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MICROSTRAINING AND DISINFECTION OF COMBINED SEWER OVERFLOWS. PHASE II

George E. Glover, et al

Edison Water Quality Research Laboratory

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Microstraining and Disinfection of Combined Sewer Overflows - Phase II

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January 1973

MICROSTRAINING AND DISINFECTION OF COMBINED SEWER OVERFLOWS - PHASE II

by

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Project 11023 FWT

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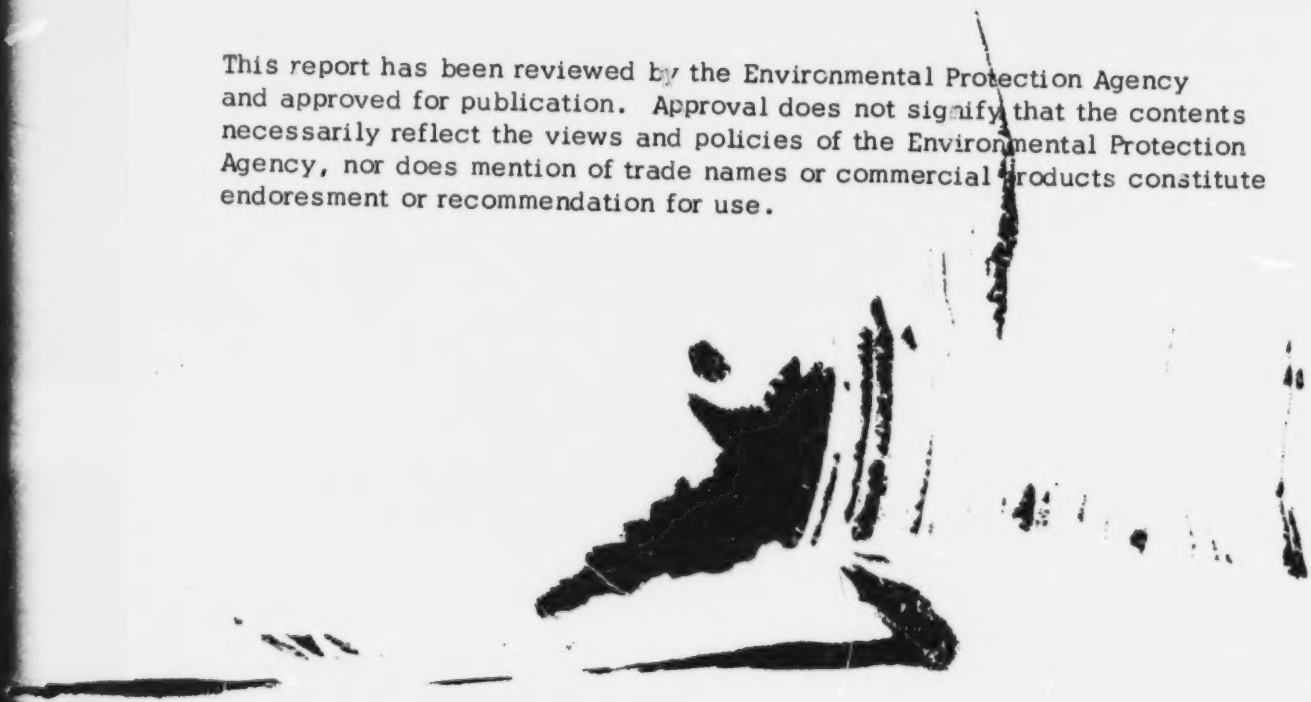

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27 Abstract A microstrainer using a screen with 23 micron apertures reduces the suspended solids of the combined sewer overflow from 50 to 700 mg/l down to 40 to 50 mg/l levels operating at flow rates of 35 to 45 gpm/ft^2 of submerged screen. The organic matter as measured by COD and TOC was reduced 25 to 40%. Coliform concentrations were 0.1 to 9 million cells per 100 ml and no reduction was brought about by Microstraining©. Storm-water service requires special analytical techniques which are described in detail. The coliform concentrations of both overflow and microstrained overflow were reduced by four or more orders of magnitude by disinfection with 5 mg/l chlorine in specially built, high rate, contact chambers of only 2 minutes contact time. The drainage area served by the combined sewer comprises 11.2 acres of a residential area in the City of Philadelphia having an average dry weather sanitary flow of 1000 gph. The overflow rates recorded were generally 100 times, with a maximum 400 times, the average dry weather flow. The extreme importance of very low - 2 minute - residence volume equipment for suspended solids removal and for disinfection in the very high instantaneous rates encountered with stormwater is shown. The cost of a microstrainer - special chlorine contact chamber installation is cited as $6,750/cfs of peak flow rate capacity less land and engineering. On the basis of 2 cfs instantaneous design overflow per acre this is $13,100/acre. ~~This report, submitted in fulfillment of Project 11023 FWT, is a continuation of the work previously reported in 11023 DAP 08/70~~. (EPA)

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ABSTRACT

A microstrainer using a screen with 23 micron apertures reduces the suspended solids of the combined sewer overflow from 50 to 700 mg/l down to 40 to 50 mg/l levels operating at flow rates of 35 to 45 gpm/ft^2 of submerged screen. The organic matter as measured by COD and TOC was reduced 25 to 40%. Coliform concentrations were 0.1 to 9 million cells per 100 ml and no reduction was brought about by Microstraining (C).

The coliform concentrations of both overflow and microstrained overflow were reduced by four or more orders of magnitude by disinfection with 5 mg/l chlorine in specially built, high rate contact chambers of only 2 minutes contact time.

The drainage area served by the combined sewer comprises 11.2 acres of a residential area in the City of Philadelphia having an average dry weather sanitary flow of 1,000 gph. The overflow rates recorded were generally 100 times, with a maximum of 400 times, the average dry weather flow.

The limitations of conventional analytical methods for organic matter in the highly variable combined sewer overflow are discussed. Modifications of the analytical methods are described.

The extreme importance of very low - 2 minute - residence volume equipment for suspended solids removal and for disinfection in the very high instantaneous rates encountered with stormwater is shown.

The cost of a microstrainer - special chlorine contact chamber installation is cited as $6,750/cfs of peak flow rate capacity less land and engineering. On the basis of 2 cfs instantaneous design overflow per acre, this is $13,100/acre.

The observed performance of the microstrainer was reduced to a permeability parameter. The influence of turbulence on chlorine disinfection rate is shown.

This report, submitted in fullfillment of Project #11023 FWT, is a continuation of the work previously reported in #11023 EVO 06/70.

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SECTION I

CONCLUSIONS

1. Suspended solids removal performance tends to confirm previous work. The microstrainer operated at flow rates of 35-45 gpm/ft^2 with differentials of 24 inches of water. At these rates, the suspended solids in combined sewer overflow were reduced from 50-700 mg/l to 40-50 mg/l and below. At the higher influent levels of suspended solids, removal performance was enhanced, yielding effluent concentrations of approximately 10 mg/l. The conventionally used, percentage removal performance criteria are not valid for Microstraining (C) of combined sewer overflow.

   Volatile suspended solids reduction paralleled the reduction of total suspended solids.

2. The highest concentration of suspended solids frequently occurs when the overflow rate is highest. The concurrence of high suspended solids concentration and high overflow rate results in a very high potential contaminant loading per unit time to the receiving stream.

   The microstrainer was unusual in that it removed a much greater percentage of the suspended solids when the concentration of these was higher.

   A reasonably sized microstrainer can then limit the pounds of suspended solids per unit time entering the stream from a combined sewer outfall. Availability of this equipment will permit the regulation of this important aspect of pollution from combined sewer overflows.

3. The organic matter in combined sewer overflow was highly variable - ranging from 10 to 2,000 mg/l as BOD_5 and 20 to 4,000 mg/l as COD and TOC. The microstrainer reduced organic matter by some 25 to 40%.

4. The total coliform content of combined sewer overflow at our site was variable from 0.1 to 3 million cells/100 ml. The fecal coliform content of the combined sewer overflow was also variable from 1,000 to 100,000 cells per 100 ml. The microstrainer demonstrated little or no ability to remove total or fecal coliform from combined sewer overflow.

5. Chlorine contact time of only 2 minutes under relatively high turbulence

conditions with chlorine dosages as low as 5 mg/l reduced total coliform from 1 million levels in combined sewer overflow down to 5 to 10 cells per 100 ml. Fecal coliform was similarly reduced under the same conditions from 100,000 levels in the combined sewer overflow down to 5 to 10 cells per 100 ml.

6. Bacteria counts made on macerated samples were generally over two times higher than on the unmacerated samples.

7. The capital cost of a microstrainer-high rate chlorination preceded by bar screening, previously reported (1), would be $13,100 (in 1972 dollars) per acre not including engineering or the 1/5,000 acre land required per acre drainage area. This capital cost is based upon 1.94 cfs design runoff per acre and upon an intensely mixed chlorine contact chamber such as used herein. The present work confirms the high rate performance of the microstrainer and shows 99.99% bacterial kills in 20 gpm pilot size, high rate chlorine contact chambers. The conclusion is repeated here that a high rate Microstraining unit and high rate chlorination are practical and the least expensive of the alternate treatment methods considered (20). The single exception is surface impounding where very low cost land is available.

8. The design criteria used for the construction of the special, high mixing intensity, pilot size contact chambers, appear to be appropriate and practical for full size units.

9. The special character of solids in combined sewer overflow (and stormwater) is vastly different from other municipal and industrial wastewaters and from the potable water sources to which a microstrainer is normally applied. The high level (liquid) differentials across the screen required to achieve the high flow rates necessary to be practical in stormwater service are far beyond those conventionally used. Thus, empirical relations and experience useful in conventional applications are misleading in the Microstraining of combined sewer overflows.

   A performance parameter was developed which is closely akin to the permeability of the deposited solids on the screen. This permeability parameter was calculated from the operating conditions of each storm. This parameter is believed to characterize most of those properties of the deposited particles which determine resistance to flow. Therefore, it can be used, for similar particles, to predict head loss at other flow rates, drum speeds and temperatures.

10. The permeability parameter was also found to increase with higher

suspended solids concentrations indicating that the additional solids in the runoff changed the properties of the average particle. Thus, with the conditions at our site, the permeability parameter can be used to predict head loss at other runoff suspended solids concentrations as well.

11. It was further found empirically that the suspended solids removal also increased as the permeability parameter increased. Conversely, then, permeability can be used to qualitatively predict solids removal.

12. The allocation of capital costs of facilities for the treatment of combined sewer overflow is not uniform from study to study. The difference in method is particularly evident when capital costs are reported as dollars-per-acre of drainage basin area. There is a broad range for the cost of the various proposed treatment methods on dollar per cu ft/sec of design through-put basis. There is also a very broad range of design runoff or sewer overflow rates specified per acre of drainage area served. The overflow rates vary from 2 cu ft/sec-acre as used in item 7 above and in our previous report (1) down to 0.2 cu ft/sec-acre for very large drainage areas where considerable flow equalization within the trunk sewers and absence of simultaneous rain fall over the entire area are considered (2). Less understandable is the use of low overflow rates such as 0.25 cu ft/sec-acre for the smaller drainage areas that might be served by satellite overflow plants (3).

SECTION II

RECOMMENDATIONS

1. Microstrainers employed in stormwater service should be operated at high differentials of approximately 24 inches water. At this differential and with the deposited (suspended) solids permeabilities we encountered, flow rates of 35-45 gpm/ft^2 can be obtained with a deep enough mat of solids to produce an effluent quality of 40 mg/l suspended solids. To achieve lower effluent levels with the suspended solids characteristic of this test period, slower drum speeds, possibly lower flow rates and/or the use of polymers will be required. Additional storms should be monitored at a broader range of conditions to confirm the possibility of cleaner effluents without polymer addition, as was illustrated in a few instances in our earlier work (1).

2. A full-scale, in-line microstrainer and chlorination facility, without an impounding basin, should be built as a demonstration plant at a site where the benefit of stormwater treatment can be measured by comparison with a previous stream quality study.

3. Additional work is required to confirm the disinfection rate on both untreated and microstrained combined sewer overflow with lower chlorine dosages, at low ozone dosages, and with low two-stage ozone-chlorine dosages.

4. In order to bring more realism to the treatment of combined sewer overflows, regulatory agencies should establish guidelines for the performance of such treatment in the many different overflow situations. In view of the range of almost two orders of magnitude in both combined sewer overflow rate per acre and in suspended solids concentrations, a generally applied percentage removal guideline would be totally ineffective in one case and excessively punitive in others. Further, in view of the rapid variations in flow rate and pollutant concentration within a given storm, as compared to the relatively steady state flow of domestic sewage, the measurement of percentage removed would be a difficult and expensive enforcement procedure. Performance guidelines should be based upon average effluent quality or stream quality or pounds pollutant added per unit time and should include residual chlorine levels.

5. In the interest of both economy and the public health, the influence of mixing intensity in the disinfection of municipal and other wastewaters should be recognized. This recognition might take the form

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of specifying a minimum mixing intensity and actual contact times in conventional chambers. Ultra high rate chambers of suitable design can be especially applicable in such cases as combined sewer overflow service.

6. With the additional potential utility of the permeability parameter to qualitatively predict suspended solids removal, it is recommended that additional storms be monitored without the use of polymers to further establish this base, and that this parameter be included as a means to aid in evaluation of polymer addition.

7. The treatment of combined sewer overflow has only recently received serious attention. Rational design of facilities to treat this very high, variable flow rate, variable composition wastewater is seriously handicapped by the use of inappropriate performance criteria and analytical methods borrowed from the relatively steady-state domestic sewage treatment art. Performance criteria should be adopted which recognize both the unusual nature of this waste and the equipment employed to treat it. For example, the mgd dimension of flow rate is frequently used because of its familiarity to sanitary engineers. It is strongly recommended, however, that they be aware that it is the instantaneous capacity of the equipment that is the governing parameter.

8. In view of the peak instantaneous capacity limitation of the equipment, the benefit of flow equalization within the sewers themselves should be fully evaluated. Also, the benefit of aboveground flow equalization basins should be evaluated to determine their capability to reduce the peak flow requirement of the equipment.

9. Similarly analytical methods should be developed and adopted which recognize the widely variable amount of and the unusual nature of the solid and dissolved matter in the combined sewer overflow. The unusual nature of the organics in combined sewer overflow is exemplified by the refractory nature of the solids and the short time interval between wetting and discharge via overflow. The BOD test is basically inaccurate, unrepeatable, and unreliable.

   The problems with the BOD test are magnified in combined sewer overflow by the probable presence of toxic metals and the difficulty of maintaining a culture of acclimatized micro-organisms for the decomposition. The use of non-biological analytical methods such as COD and TOC for combined and separate sewer overflows is recommended.

   The maceration pretreatment improves the COD as well as the BOD_5 methods and should be adopted.

Also, the effect of cell clumping, occlusion of cells in solids particles, and delayed growth character of recently wetted spores strongly indicates the use of the maceration pretreatment of sample prior to cell counting methods for truer bacteria counts. The maceration technique is of recent origin and additional work is required to define and optimize the several parameters - particularly duration of maceration and mixing intensity.

10. Additional storms should be monitored to determine the advantage of the use of polymers in suspended solids and organic removal from combined sewer overflow. Particle size data would be helpful in such an effort.

SECTION III

INTRODUCTION

This work is a continuation of the study of a commercial size microstrainer, with chlorination and/or ozonation, treating combined sewer overflows. The original work was reported as "Water Pollution Control Research Series", Report #11023 EVO (1) and in abbreviated form elsewhere (4).

Previous Work

Briefly, the original work covered 26 storms during the period 1/69 to 9/69 from an 11.2 acre residential drainage area in Philadelphia, Pa. A Crane-Glenfield microstrainer fitted with 47 sq ft of first Mark I (35 micron openings) screen and later with Mark 0 (23 micron) screen was used. The suspended solids content of the storm overflow ranged from 20 to 500 mg/l, more or less proportional to the storm overflow rate. The flow rate through the microstrainer was controlled at a variable rate proportional to the storm overflow rate and average flow rates ranged from 1 to 45 gpm/ft^2 of submerged screen area. In order to achieve the higher flow rates through the screen with existing pumping equipment, 4/5 of the screen area was masked during the later tests with Mark 0 screen.

About 3/4 of the storms were unattended. The Microstraining equipment idled continuously and the drum speed was adjusted at the onset of storm-water flow by automatic controls. The performance was monitored by recording instruments and by flow insensitive composite samplers. The outlet weir level was set initially for a 6 inch of water maximum differential and for the later storms the weir was set for 24 inches of water.

The suspended solids removal ranged from 20% to 98% with the higher removals generally occurring at the higher flow rates and always at the higher inlet suspended solids concentrations. Removal of suspended volatile solids (SVS) paralleled the removal of suspended solids (SS). The microstrainer performance, as opposed to other treatment techniques, was insensitive to wide and rapid changes in the composition of the combined sewer overflow (e.g., the content of suspended solids or the temperature).

The BOD_5 of the sewer overflow ranged from 5 to 500 mg/l averaging 50 mg/l. The sanitary contribution was less than 2,000 gph from this area while measured storm overflows were frequently over 100,000 gph. BOD_5 removals by the microstrainer ranged from 80% down to 10% on 16 of the storms. On the other 10 storms, increases in BOD were reported and many were of significant magnitude.

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Similarly, the total and fecal coliform counts on the sewer overflow ranged from 0.1 to 3 million cells per 100 ml. Coliform removals by the Microstraining unit ranged from 90% down to 20% on 16 storms while on the other 10 storms there were increases in coliform count. Many of the increases were significant and, incidently, did not occur altogether on the same storms as the BOD increases.

Bench scale chlorination of the microstrained combined sewer overflow showed some 99% bacterial kill with 5 mg/l dosage and 10 minute contact time. Ninty-nine percent kill was also obtained in one test in 0.5 minute contact time using 15 mg/l chlorine. Ozonation in 20 gpm pilot scale equipment using 4 mg/l ozone dosage and 12 minute contact time showed some 99% bacterial kill.

An economic study was made of the cost of combined sewer overflow treatment by a Microstraining filter and disinfection as well as treatment by other proposed methods. This study (1) indicated that all methods, except for surface impounding which on a design basis of 2 cfs runoff per acre surface impounding requires 1 acre basin area of 10 ft depth for each 30-50 acres drainage area, would require a capital cost of at least $10,000/acre excluding land and engineering (1969 dollars). The other methods included some proposed (14) treatment trains comprised of conventional and advanced sewage treatment processes such as screening flotation, bio disc contact, activated carbon, primary clarification, etc.

This study indicated that the Microstraining filter and disinfection at high rate was the least expensive and most compact of the other methods then proposed and, moreover, required less than 1 acre of treatment area per 5,000 acres of drainage area.

Present Work

The second phase of the work was conducted to confirm the performance of the Microstraining unit and disinfection at high rate under more closely controlled conditions. The same facility was used. It was adapted by the addition of a third microstrainer feed pump, and by the substitution of automatic vacuum type discrete samplers (Serco Model SG-15 manufactured by Sanford Products) and the necessary flow-through sample chambers for the composite sample pumps, which withdrew directly from the microstrainer inlet and outlet chambers. Also, two specially-designed chlorine contact chambers of the flow-through type were substituted for the batch type glassware previously used. The 5 ft diameter by 3 ft microstrainer with 9.4 ft^2 of open Mark 0 (23 micron) screen was used as before. The inlet and outlet chambers were reduced in size to reduce residence time within these chambers.

The controls were changed so that only storms producing more than 0.66 mgd (30,000 gph) of overflow were treated (30 times mean dry weather flow). The flow rate through the microstrainer was preset and maintained at that constant rate during a storm, independent of variations in sewer overflow rate.

The disinfection procedure used in the second phase was developed to confirm the economically attractive high rate chlorination work of the first phase. Also, the procedure was designed to confirm or disprove the postulated improved ease of disinfection of the microstrained water. To this end the raw overflow as well as the microstrainer effluent were disinfected simultaneously in separate contact chambers during an attended storm. The chlorine dosage of the raw water was set higher to compensate for the greater chlorine demand so that both raw and microstrained water flows had the same residual after 195 seconds.

Project time and occurrence of suitable rainfall have not permitted expansion of the original data for disinfection via ozonation or with both ozone and chlorine.

SECTION IV

EXPERIMENTAL EQUIPMENT

Microstrainer

The test system uses a Microstraining filter for the removal of suspended solids followed by sodium hypochlorite disinfection. The microstrainer is a rotating drum filter, 5 ft in diameter by 3 ft long, covered with specially woven Mark 0 stainless steel wire fabric having nominal openings of 23 microns.

Eighty percent of the screen area is blanked with three layers of polyethylene sheet so that 9.4 sq ft is open and effective in the straining process.

The water enters the drum through the open end, flows radially through the drum into the outlet chamber and deposits suspended solids on the inside of the drum screen (see Figure 1). As the drum rotates, the dirty screen passes up under backwash jets which wash the deposited solids off the inside of the screen into the wash water hopper and then to waste. The mat of deposited solids on the screen causes the resistance to flow to increase; i.e., causes the differential to increase for any flow rate through the unit. The mat of deposited solids are a much finer filter than the screen itself and the thicker deposits improve the removal of suspended solids.

For the purpose of monitoring the unit's performance on short duration storms, the inlet and outlet chambers were reduced to minimum volume to reduce response time. The sample streams were withdrawn from the inlet chamber at the entrance to the drum and from the outlet chamber at the center of the open screen panel about 2 inches from the bottom of the drum; i.e., at mid-point of screen travel during submergence.

The microstrainer ran at its idling speed continuously, under UV light, with backwash jets flowing city water during dry weather. During storms, the jets could have been supplied with microstrained water by means of a pump, but city water was used here too for convenience. The backwash flow was less than 2 gpm (less than 1/2% of combined sewer flow through the microstrainer). Normally, the small (approximately 1-1/2 times the mean dry weather) flow would be directed to the interceptor sewer downstream of the overflow regulator but for our convenience, it was directed to the creek. At the onset of a storm, automatic controls started the feed pump(s) when the overflow rate reached 0.66 mgd (30,000 gph). The selection of 0.66 mgd (30,000 gph) as the trigger flow rate was to avoid

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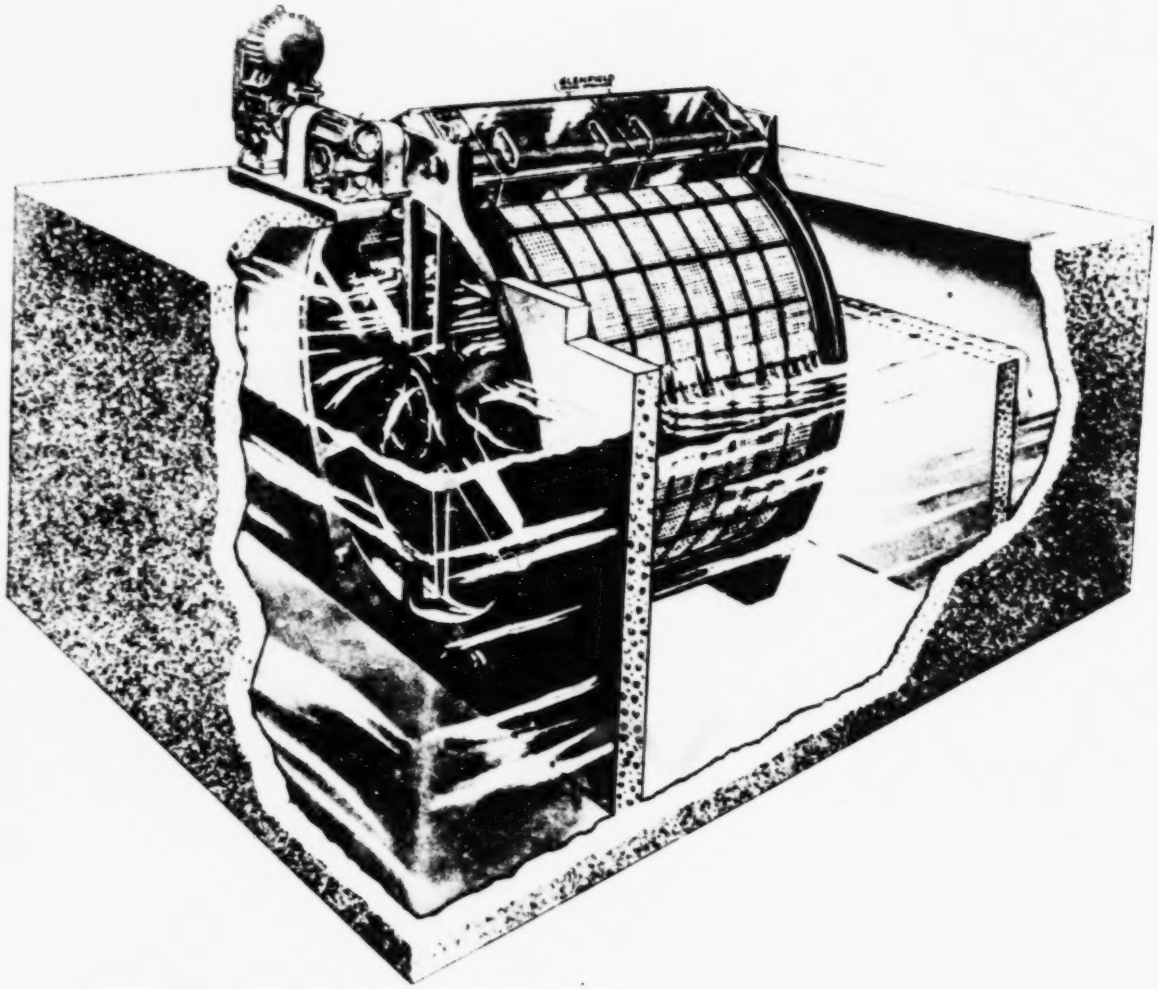

Figure 1

Isometric Drawing of a Microstrainer

storms of insufficient intensity or duration to yield meaningful results.

The flow rate of water to the microstrainer was controlled at a constant rate, predetermined by the selection of the number of pumps to be activated by the storm overflow rate control. The flow rate through the microstrainer was then constant and independent of the combined sewer overflow rate. The flow of water through the microstrainer caused a difference in level between inlet and outlet chambers. The automatic drum speed control sensed these levels and increased the drum speed proportional to the differential. The proportional control band maintained the drum speed at a maximum (6 rpm) speed with a maximum differential of 24 inches of water and proportionately slower speeds at smaller differentials. It might be noted here that the preferred mode of operation for continuous operation is to control drum speed to maintain the maximum differential of 24 inches by use of a reset-proportional band. This mode of control was abandoned during the previous phase of this work on intermittent flows of stormwater because of the effect of so-called "re-set windup" during long idle periods between storms.

The automatic sampler controls were also triggered by the overflow rate control. The inlet sampler was delayed 2 minutes and the outlet sampler was delayed 5 minutes after the start of the feed pumps. The residence time between inlet and outlet sample points was 3 minutes, and plug flow was obtained to a remarkable extent as indicated by salt tracer studies to be described in Section V, SAMPLING.

Chlorine Disinfection Equipment

The chlorine disinfection equipment consisted of two specially-designed chlorine contact chambers each equipped with a Schulte and Koerting, variable area, rate-of-flow meter for the inlet water, a Lapp, variable capacity, hypochlorite, metering pump, a container of hypochlorite solution and a homemade inorganic cation exchanger.

The equipment was manually operated and one of the chambers - the one treating the microstrainer effluent - was equipped with a feed pump. The microstrainer inlet pressure was sufficient to supply the other chamber. The field test equipment used to monitor and control the disinfection conditions consisted of an orthotolidine comparator set for free and combined available chlorine, a millivolt meter with oxidation-reduction potential (ORP) cells, a pH meter, and a liquid level gauge.

The contact chamber shown in Figure 2 provided 4 minutes residence (contact) time at a flow rate of 20 gpm. The large number of baffles insured true plug flow so that full residence time was utilized for chlorine contact.

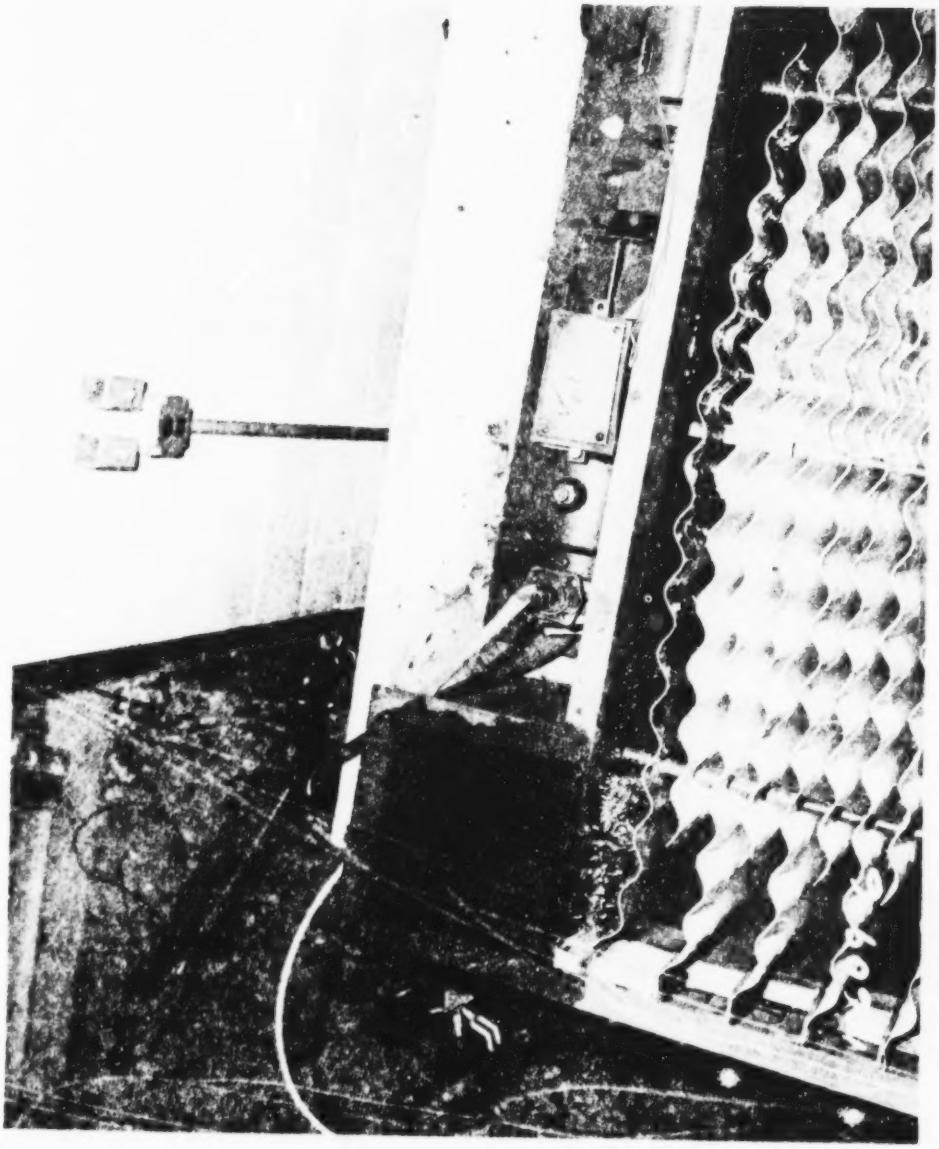

Figure 2

Intensely Mixed 4 Min.
Chlorine Contact Chamber

The large number of baffles resulted in narrow passages and high liquid velocity. The high liquid velocity (.27 ft/sec) together with the corrugated configuration of the baffle panels resulted in a head loss of 1-1/2 inches over the 64 ft long travel of liquid. This energy dissipation created a relatively high velocity gradient throughout the liquid.

Sample taps at 1/4, 1/2 and 3/4 the length of the liquid path provided samples at 75, 135, and 195 sec contact time, respectively at 20 gpm. The top of the chamber was open so that samples could be collected at the surface after the inlet chamber (15 sec) and at the end of the path (255 sec).

All disinfections were performed at a constant flow rate of 20 gpm through the chambers. Except where it was not possible, both the raw combined sewer overflow (microstrainer influent) and the microstrained effluent were disinfected simultaneously during the storm. A pre-selected initial concentration (dosage) of chlorine was controlled by the setting of a positive displacement Lapp pump and by the concentration of the hypochlorite solution. No attempt was made to control the residual concentration of the microstrainer effluent stream. However, it was measured after 195 seconds contact and the dosage to the raw combined sewer overflow with its higher chlorine demand was adjusted to yield the same 195 second residual. No attempt was made to control pH; however, since the combined sewer overflow was already in pH 7-8 range and the hypochlorite was alkaline, the hypochlorite pump discharge was passed on later storms through an inorganic cation exchanger in hydrogen form to avoid raising the stream pH to a higher range favoring less effective forms of chlorine. The cation and alkalinity content of the hypochlorite solutions and the buffering action of the combined sewer overflow were such that instead of raising the pH about .3 - .4 without the cation exchanger - the pH was lowered - .4 - .5 units.

After equilibrium was established as indicated by constancy of pH and ORP at all points, and equal chlorine residuals at the 195 second sample points, samples were taken from the flowing sample cocks and from the surface of the flowing liquid at locations representing 15, 75, 135, 195, and 255 seconds contact time. Samples for bacteria analysis were collected in sterile bottles containing several ml of sodium thiosulfate solution to quench the chlorine and were shaken vigorously and covered.

Chlorine residuals were measured with the orthotolidine comparator set. The comparator equipment was arranged to permit the analysis to be performed in about 15 seconds. With the necessarily short reaction time between sample and the orthotolidine reagent, the Standard Methods Procedure indicates that for practical purposes only free chlorine was

measured, although at the recommended 5 minute reaction time the orthotolidine will reveal both free and combined chlorine.

The hypochlorite stock solutions were made up based upon the label concentration of household bleach solution and stored until needed.

In one case, the actual concentration of the hypochlorite stock solution was measured at the time of use. The calculated chlorine dosage based upon this measurement was within a few percent of the preselected dosage.

SECTION V

SAMPLING

The inconsistencies in BOD and bacterial results noted in the first phase of this work (1) prompted a greatly expanded analytical program and alterations in the sampling procedures for this the second phase of work. Unfortunately, many storms occurred and were sampled before any analytical results were reported by August 24, 1971, at which time it was evident that there were faults in the sampling and/or analytical procedure. It was then found that the automatic sampler bottles were being incorrectly sterilized in the field. Accordingly, neither the organic content nor coliform results prior to October 10 when the automatic sampler was involved, are considered in any of the discussions.

Prior to October 10, the sample bottles placed in the automatic vacuum sampler (Figure 3) were sterilized with methanol and were not completely dried. At the same time, the procedure for sterilizing and cleaning the sample tubes was revised to include flushing the individual sampler tubes with hypochlorite followed by a high velocity rinse with city water.

Many of the storms were of short duration and all were of varying, sometimes rapidly varying, composition. The response time of the automatic sampler and its auxiliary flow-through chamber to changing composition of the flow through the microstrainer is important. As previously mentioned, the volume of the microstrainer inlet and outlet chambers had been reduced to improve response time. The microstrainer inlet sample was withdrawn from a point in the inlet chamber 2 inches axially out from the inlet end of the drum and about 18 inches radially to the emerging side of the drum center.

The inlet to the effluent flow-through sample chamber was located at the center of the open screen panel about 2 inches from the bottom of the drum. Figure 4 shows the microstrainer effluent chamber with the plywood wall reducing the chamber volume and with three sample points. The visible opening collects from a point near where the screen enters the water. The sample chamber inlets are then located less than 2 seconds from the inlet and outlet of the drum. The flow rate through the sample chambers was approximately 10 gpm, at which rate the 6.5 and 13 gallon volumes of the influent and effluent flow-through chambers could cause a response lag of .65 and 1.3 minutes respectively.

Figure 3 shows the effluent sampler with 24 evacuated bottles; the 13 gallon flow-through sample chamber and the sample pipes from the microstrainer outlet chamber.

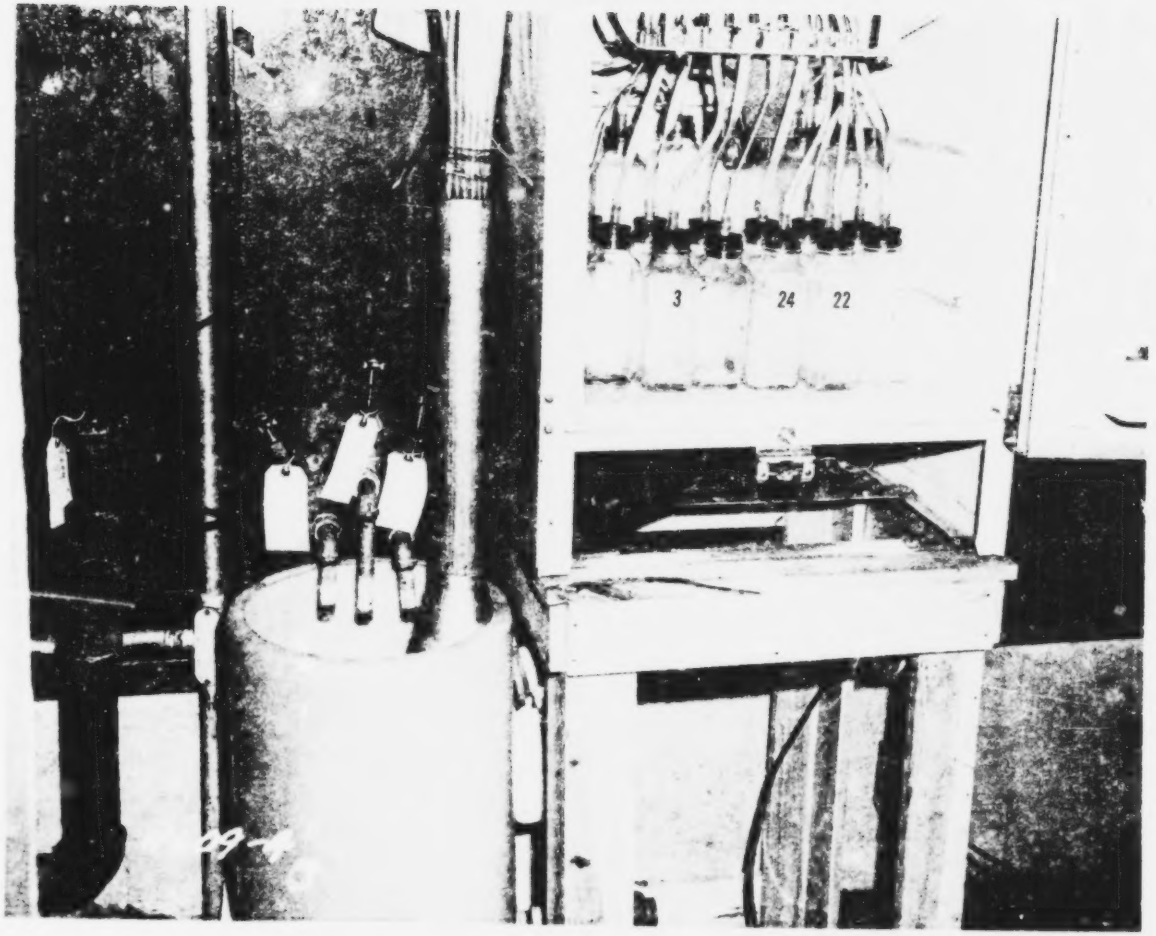

Figure 3

Discrete Sampler With Flow-tnrough Chamber

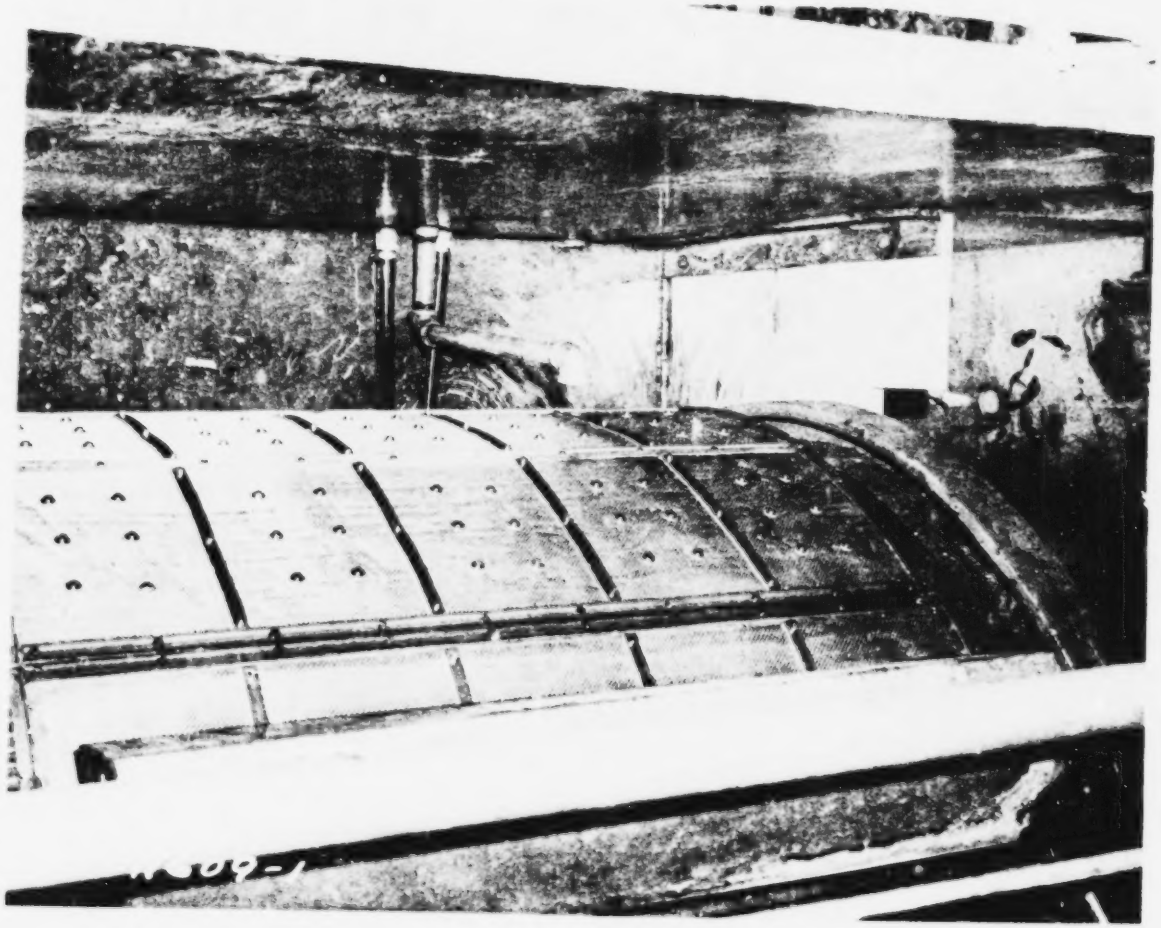

Figure 4

Outlet Chamber of Microstrainer Showing
Three Effluent Sample Lines

The cumulative effect of all these contributions to the time lag in the response of sample composition to changing overflow water concentration is shown by a typical salt tracer test (Figures 5 and 6).

Note that Figure 5 shows a lag of 5 minutes - the theoretical residence time of the volume between the pump suction, inlet line, inlet chamber, drum volume and the outlet chamber. Similarly, Figure 6 shows a lag of 2 minutes - the theoretical residence time of the volume between pump suction, inlet line, and inlet chamber.

Figure 5 and Figure 6 also show that there was surprisingly little back mixing (departure from plug flow) through the microstrainer. Also, these curves show that about 1 to 1-1/2 minutes additional lag was introduced by the flow-through the sample chambers.

The inlet sampler was started automatically 2 minutes after the feed pumps (flow of water) were started at the beginning of a storm. Similarly the effluent sampler was started 5 minutes after the pumps started, so that the first sample taken of inlet and outlet represented more or less the same increment of water. The samplers then collected instantaneous, discrete 300 ml samples of the two flows every 2 minutes and stored them in a refrigerated chamber.

The full analysis program for composite samples required approximately 2,700 ml sample volume which is the contents of at least nine automatic sampler bottles. Where sufficient number of samples were collected both a composite sample covering a period of the storm and discrete samples were analyzed.

In general, composite samples were made up by plotting the collection time of the instantaneously collected sample increments on the storm overflow rate meter chart and selecting those increments representative of a period of more or less uniform storm overflow rate.

The sampling procedure used in each case is noted in an abbreviated manner in Table 2. For example, the "8 minute automatic composite" notation for the storm of 7/1/71 indicates that the sample was composited from the contents of 4 or 5 instantaneous samples collected over an 8 minute period, beginning 5 minutes after the onset of the strom at 7:55 p.m. The "hand discrete" notation for the storm of 7/19/71 at 11:11 p.m. indicates that a large volume sample was collected by hand at time 11:11 p.m. and was analyzed (in addition to the sample composited from the automatically collected instantaneous samples over an 18 minute period during the 10:33 p.m. to 11:11 p.m. storm).

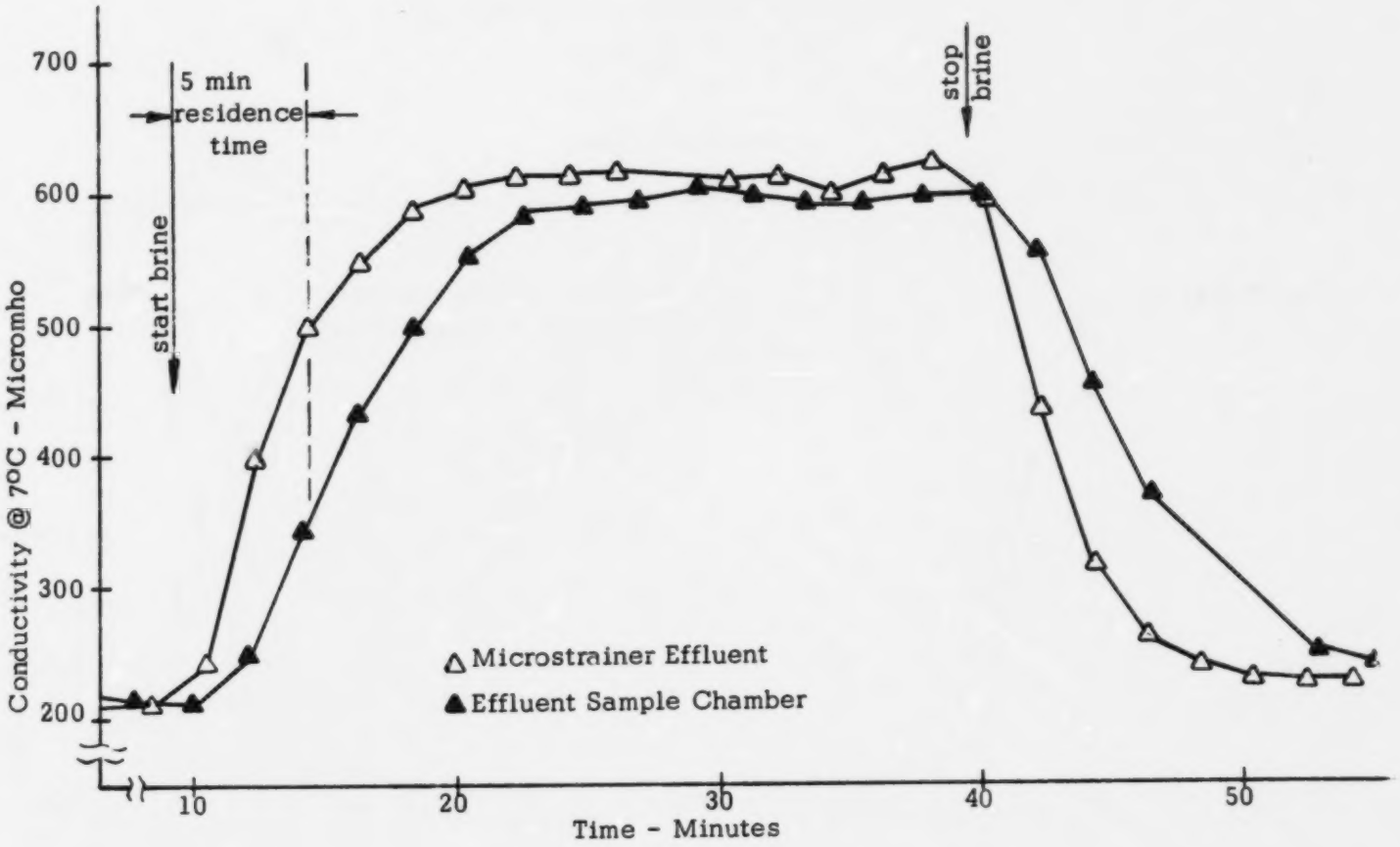

Figure 5

Salt Tracer Test - Microstrainer Effluent

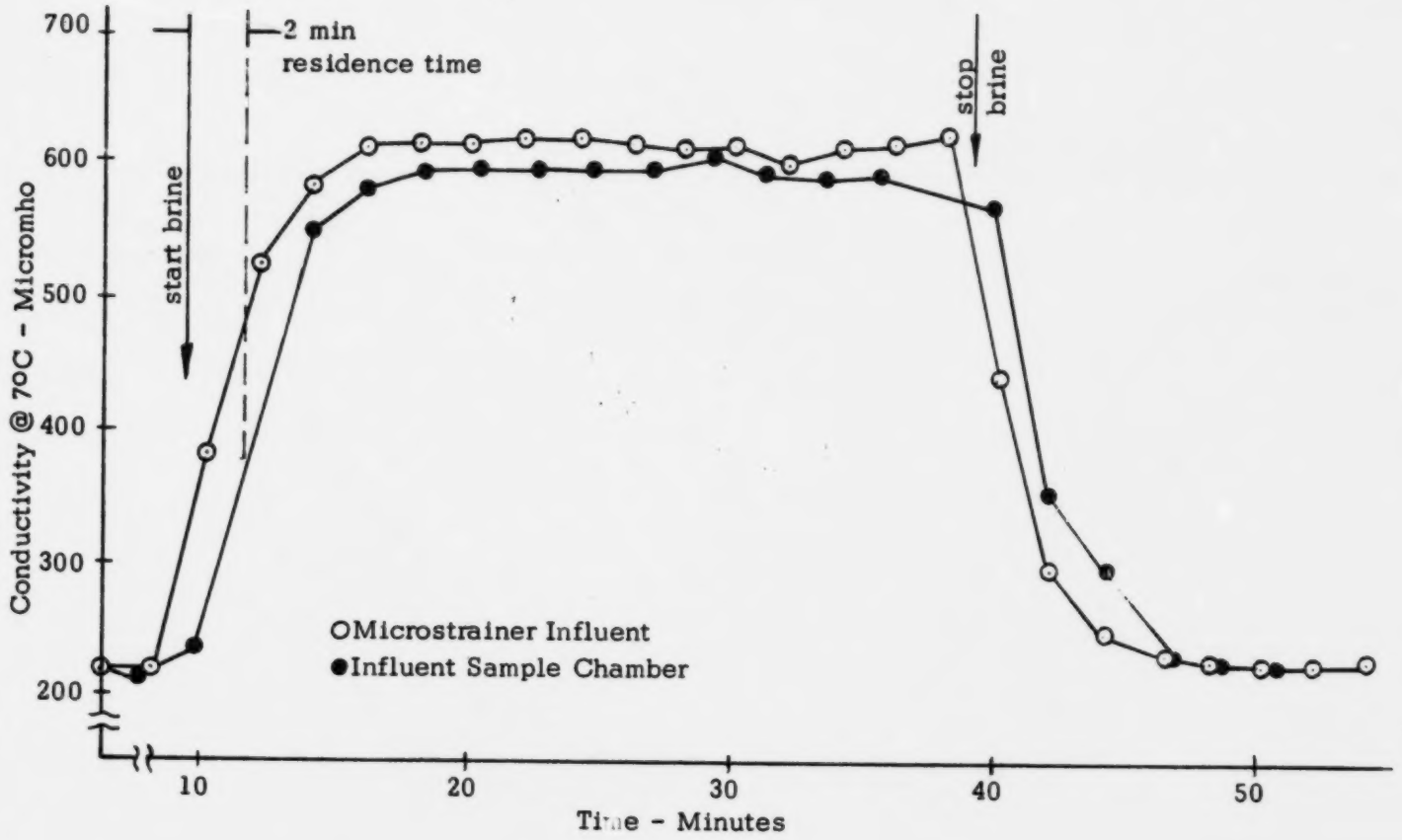

Figure 6

Salt Tracer Test - Microstrainer Influent

The "auto discrete" notation for the storm of 7/30/71 indicates that the automatic samplers collected an insufficient volume during this short duration storm and the collected volume was submitted for analysis under the abbreviated program for discrete samples.

In some cases, due to a failure of one of the samplers, a composite of the bottles of the working sampler was submitted for analysis along with a large sample of the contents of the flow through sample chamber of the faulty sampler collected long after the storm. These samples almost always yielded anomalous results and were discarded in the Discussion of Results Section.

SECTION VI

ANALYTICAL PROCEDURE

The anomalous increases in BOD and coliform counts on passage through the microstrainer noted in the first phase of this work prompted a greatly expanded analytical program in this second phase.

The additional analytical treatment had two general objectives. The first was to get more reliable data on microstrainer performance. The second was to determine if the Microstraining operation had an effect on the combined sewer overflow that might influence the results of conventional analytical methods.

It was postulated (1) that the anomalous increases in BOD_5 and coliform count observed on passage through the microstrainer reflected a significant change in the character of the particulate matter by the Microstraining process.

Specifically, it was postulated (1) that the organic matter in particulate form was reduced in size so that it became more available to biological oxidation during the period of the 5 day BOD test. To investigate this possibility in depth a manifold approach was adopted. Long term oxidation rate analyses were made so that the ultimate BOD could be predicted. Further, the suspected effect of particle size reduction by Microstraining was simulated by maceration of both influent and effluent samples in a Waring blender prior to the BOD test. Finally, the BOD_5 test was supplemented by the more accurate COD and TOC tests for organic matter.

Similarly, it was postulated that many of the coliform in combined sewer overflow existed in clumps which were broken up by passage through the microstrainer and were counted as individual cells in the effluent sample. To explore this possibility, the particle size reduction effect of the microstrainer was simulated by maceration of an aliquot of both influent and effluent samples in a Waring blender.

Also, it was postulated that passage through the microstrainer would so reduce particle size that many of those cells shielded by inclusion in a particle of suspended solids would be exposed to disinfection. To explore the possibility that the Microstraining resulted in greater ease of disinfection a two-fold approach was adopted. The combined sewer overflow both before and after Microstraining were subjected to chlorination simultaneously in identical chambers. The coliform counts were made on the chlorination study samples after maceration in a Waring blender to normalize

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any difference in particle size of samples before and after Microstraining.

The maceration technique recommended by Geldrich (5) and the other analytical procedures are described in Appendix A. The apparent effects of the maceration operation on the BOD, COD and coliform analyses are described in the Discussion of Results, Section VII, Organic Removal by Microstraining.

SECTION VII

DISCUSSION OF RESULTS

Drainage Area - Rainfall and Runoff

The 11.2 drainage area shown in Figures 7 and 8 is served by a combined sanitary and storm sewer. Rainfall Intensity-Return Frequency curves for the drainage area, as furnished by the Philadelphia Water Department, are shown in Figure 9. The drainage area can be characterized by a maximum calculated runoff coefficient of 61% using the rational method. The dry weather sanitary flow is shown in Figure 10.

Table 1 shows the hydrographic characteristics measured for 19 storms in 1971 and 1972. Rainfall intensities were measured at a rain gauge located in our test drainage area, about 100 yards from the test site. The rain gauge was a 4.8 inch dual traverse, 6 hour Universal Rain Gage, manufactured by Belfort Instrument Co.

The flow through the slot-like regulator to the interceptor sewer was approximately 0.35 mgd for all storms based upon the observed opening and a calibration curve for this particular regulator furnished by the Philadelphia Water Department. (Note that in contrast to normal practice the interceptor flow was unintentionally 15 times the average dry weather flow.) The excess combined sewage overflows into a concrete trough in which are located the feed pumps of the experimental combined sewer overflow treatment facility. During a storm these pumps feed a part of the overflow through the treatment facility and the remainder of the overflow bypasses the facility and falls over a combination trapezoidal-rectangular weir directly to a creek. The trough and weir impounds approximately 4,000 gallons.

The flow rates into the treatment facility and the flow rate over the weir are continuously measured and recorded. The storm flow noted in Table 1 is the sum of the bypass flow plus the flow through the treatment facility plus the constant 0.35 mgd flow through the regulator minus the dry weather sanitary flow. The storm flow listed in Table 1 was the peak 5 to 10 minute flow observed during the indicated duration. The average storm flow, at least during the shorter storms, during microstrainer operation was within 0-20% of the peak flow listed. As previously mentioned the microstrainer operated at a constant flow rate during any storm although the overflow rate varied greatly.

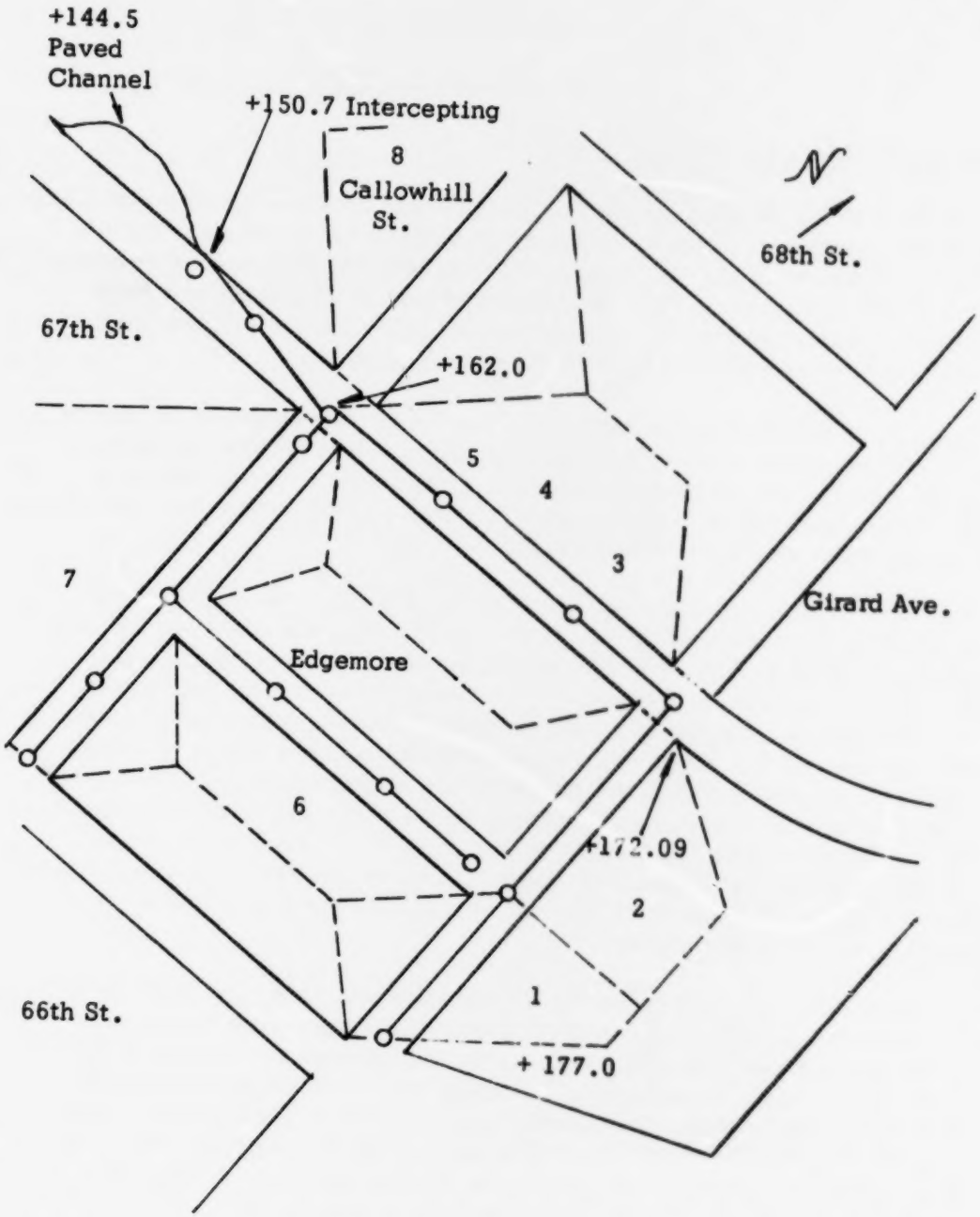

Figure 7

Outfall
67th & Callowhill

Figure 8

Drainage Area

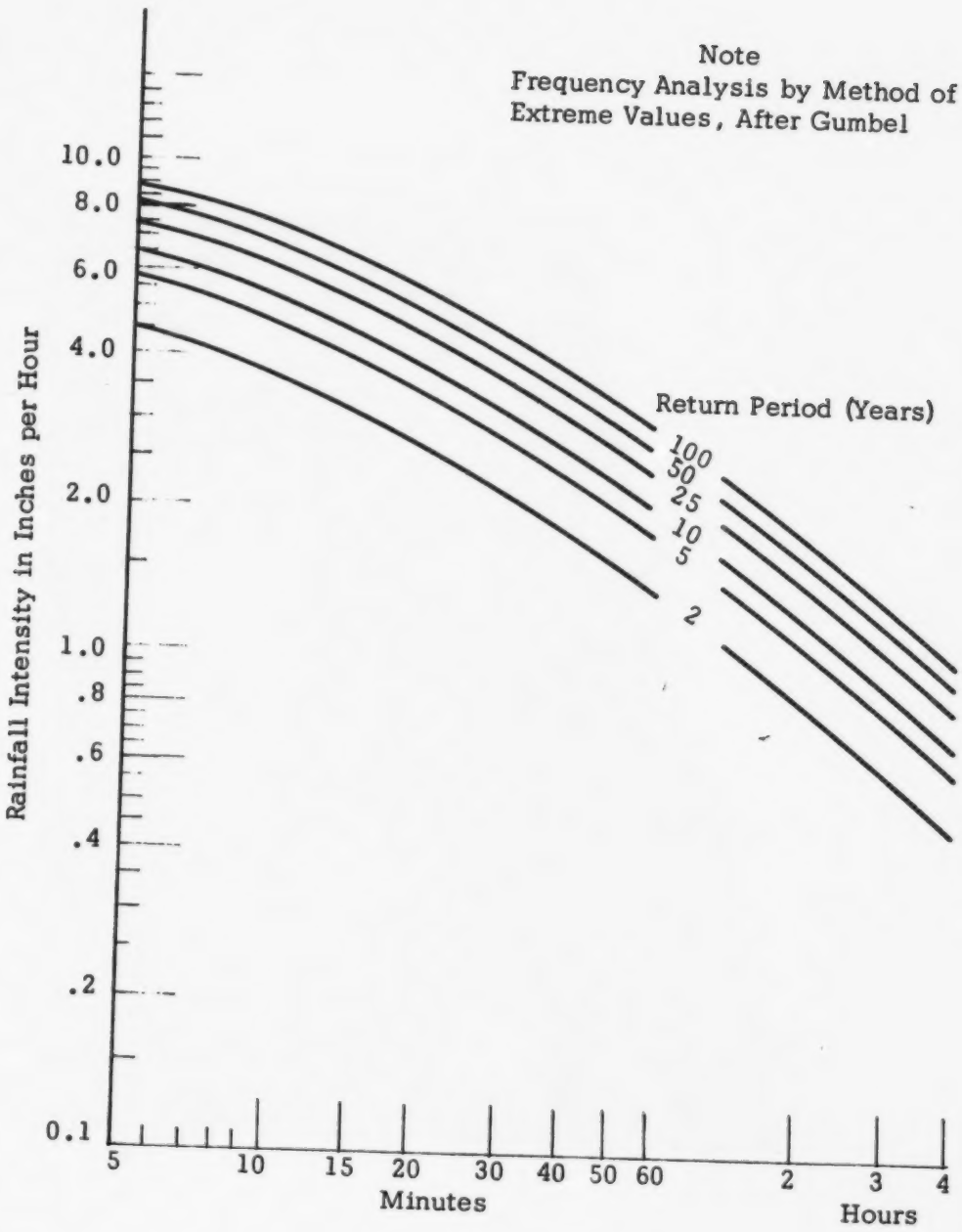

Figure 9
Rainfall Intensity - Philadelphia, Pennsylvania
1903-1951

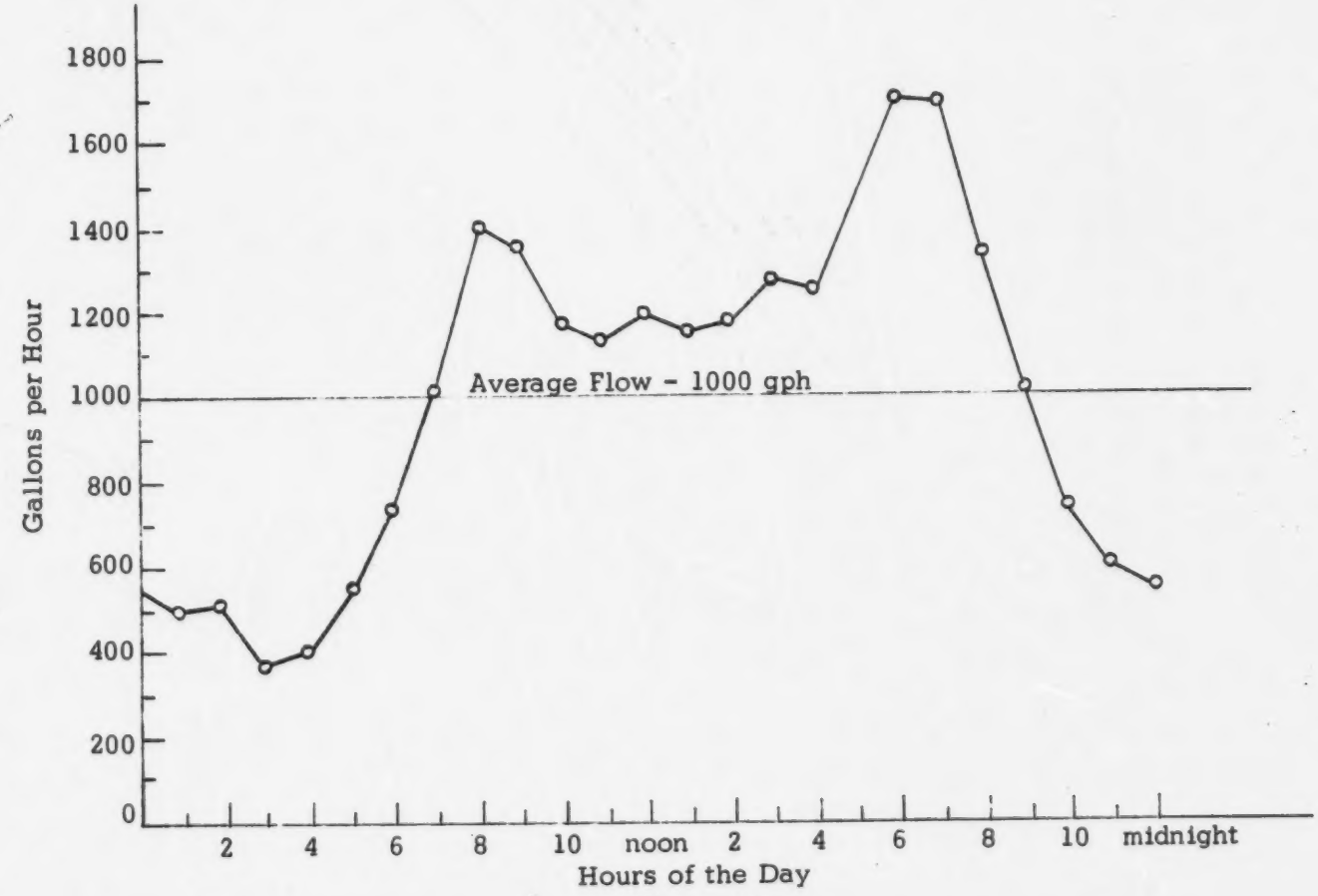

Figure 10
Hourly Variations, Sanitary Flow, 67th & Callowhill Sts. Sewer
(City of Philadelphia Data)

Table 1

Hydrographic Data

Date	Time Start	Time Stop	5 Min Intensity in/hr	10 Min Intensity in/hr	Storm Flow MGD
7-29-71	7:10p	7:24p	1.80	1.68	2.3
7-30-71	11:15a	12:17p	1.33	1.14	4.4
7-30-71	12:44p	12:57p	0.96	0.72	2.8
7-31-71	12:18a	12:31a	1.20	0.65	2.8
8-27-71	11:35a	12:36p	1.80	1.50	3.2
9-10-71	5:34p	5:52p	1.80	1.32	4.8
9-11-72	7:43a	8:05a	2.32	2.10	2.9
9-11-71	9:00a	9:36a	2.39	1.65	2.8
9-11-71	9:50a	10:56a	4.67	4.37	10.7
9-11-71	6:14p	6:37p	2.45	2.04	2.8
9-13-71	12:22p	12:30p	0.84	0.84	1.5
9-13-71	1:41p	2:35p	1.80	1.20	5.0
9-21-71	2:56a	3:08a	2.04	1.20	2.5
9-21-71	3:34a	3:53a	2.82	1.95	3.1
10-10-71	6:38a	7:37a	0.75	1.03	1.9
10-10-71	10:08a	10:40a	0.96	0.66	1.5
11- 2-71	7:23p	7:41p	1.80	1.44	2.8
11-29-71	5:25p	5:47p	0.84	0.78	2.4
2-13-72	1:18p	1:44p	0.60	0.48	1.2

Table 1

Hydrographic Data

Actual Runoff Coefficient	S.S. Influent mg/l	Permeability Parameter gpm/ft^2 x cp	BOD_5 Influent mg/l	Total Inf Coliform million cells/100 ml	Inf Fecal Coliform
.185	27	-	30	2.80	.0740
.530	115	.00164	260+	N.D.	.0010
.534	39	.00127	N.D.	N.D.	N.D.
.588	52	.00051	N.D.	N.D.	N.D.
.294	29	.00116	880	0.85	.0023
.500	190	.00086	820	0.28	.0840
.188	90	.00158	2240	8.40	.0500
.229	130	.00258	2280	8.70	.0060
.334	80	.00066	1100	2.30	.0270
.185	96	.00133	3300	0.88	.0120
.248	231	.00335	2150	N.D.	N.D.
.576	72	.00091	1730	0.53	.1330
.292	33	.00042	1620+	N.D.	N.D.
.220	64	.00124	885	0.46	.0030
.258	751	.01252	1910	0.20	.0068
.314	14	.00101	8	0.90	.0170
.265	357	.00465	55	0.32	.0040
.422	115	.00192	13	0.57	.0470
.349	208	-	-	0.07	.0640

ND - Not Determined

The actual runoff coefficients listed in Table 1 are based upon the 5 to 10 minute peak overflow rate and the maximum 10 minute rain intensity occurring during a period 30 minutes before microstrainer started and 15 minutes before it stopped.

Measurements of the suspended solids, BOD_5, total coliform and fecal coliform characterized the quality of the combined sewer overflow. Calculation of the permeability of the solids after deposition on the screen yielded an overall characterization of the type of solids, including such particle properties as average particle size, breadth of the range of particle sizes, the shape of particles and their rigidity.

Figure 11 shows combined sewer overflow versus the 10 minute storm intensity. Guidelines showing runoff coefficients of 0.1 and 0.6 are shown for convenience in interpreting the data. The line shown on Figure 11 represents the best straight line fit of the storm flow and rain intensity data as obtained by regression analysis. However, any factor which influences the ability of a surface to accept rainfall (such as rainfall pattern, rainfall intensity, geography, antecedent dry period, etc.) must affect the runoff coefficient.

The relationship between suspended solids content and overflow rate is shown in Figure 12. The data here is much too diffuse to draw any conclusion. However, the possibility of high suspended solids content occurring at high overflow is strongly indicated. No explanation is available for the scatter of the data during these late summer and fall storms. In the previous phase of our work during winter and early summer of 1969, the reported (1) relationship was definitely that of increasing suspended solids content with increasing overflow rate. In any event, it is probable that the higher suspended solids will sometimes occur at the higher overflow rates and the resulting high solids rate must be considered in equipment selection.

The practice of taking discrete samples, when possible, during a storm was adopted even though the major source of information regarding the character of the untreated and treated combined sewer overflow was to be a composite sample. The purpose of the discrete samples was to obtain additional information on the change in composition of combined sewer overflow within a storm. The storms were of such short duration that it was generally impossible to obtain more than one discrete sample because of the press of other duties. A single discrete sample, by itself, or in conjunction with a composite sample could not and did not accomplish the secondary objective of characterizing the variation in composition of the overflow during a storm.

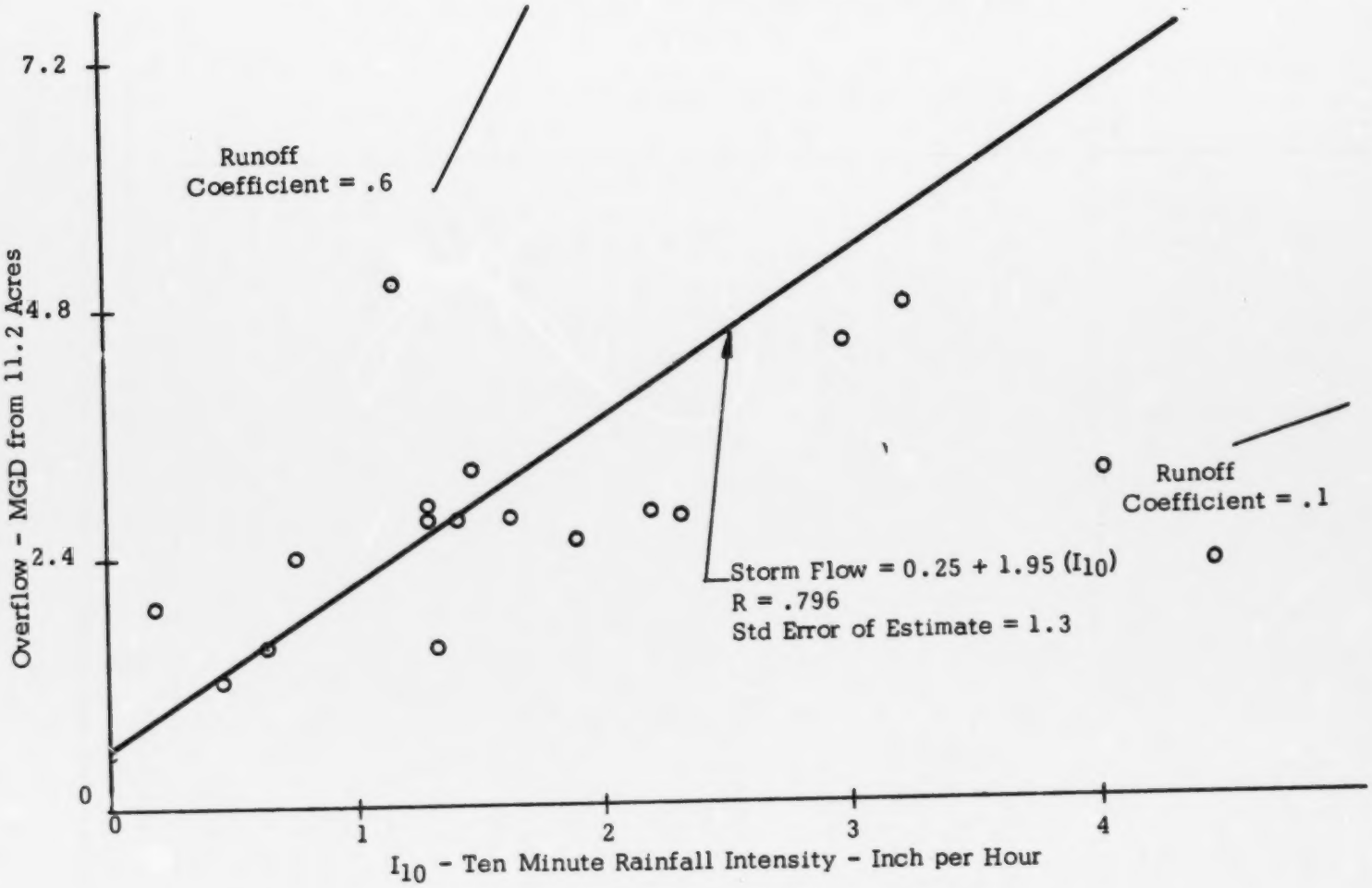

Figure 11
Observed Runoff Coefficient Based Upon 10 Minute Intensity

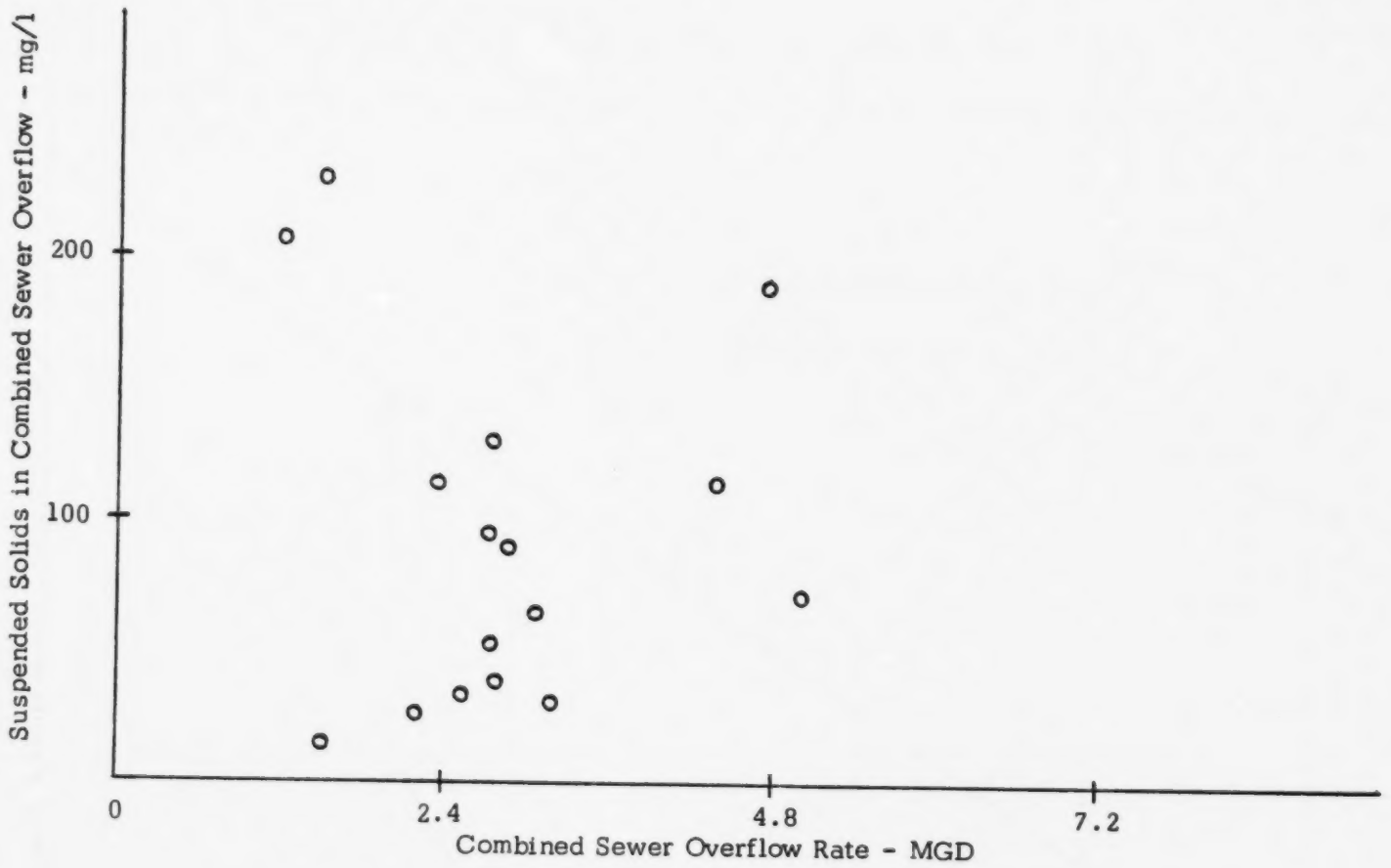

Figure 12
Suspended Solids in Overflow vs Overflow Rate

It appears that the characterizing of the variation in composition of the overflow during a storm cannot be achieved by a catch as catch can schedule imposed upon an already busy program. To accomplish this end only the discrete sampling program must be carried out on a number of storms. It is suggested that this work be undertaken as a separate study.

The Importance of Differential

The screen area available for straining is determined by the water level inside the drum. This level is kept at least an inch below the lip of the backwash water hopper but otherwise as high as possible. A maximum differential is predicted or selected and the water in the outlet chamber is maintained, by the position of the outlet weir, at such a level that the water inside the drum will not overflow the washwater hopper at the differential anticipated at the design flow rate.

In conventional applications such as the straining of algae from lake water, the flow rates are kept low to avoid shearing fragile particles through the screen, and design differentials are about 6 inches of water (6).

The physical conditions of our tests are shown on Table 2.

For combined sewer overflow service, the design differential used is 24 inches of water. The differential varies directly as the radial flow rate (gpm/sq ft of submerged screen), the viscosity of the water, the weight of solids retained (i.e., the thickness of the mat) and with the characteristics of the solids. It might be noted here that the head loss of clean water at 58° F through a clean Mark 0 screen at 45 gpm/sq ft is less than 5 inches of water. The additional differential is due to the resistance to flow of (a) the interaction of screen and solids, and (b) the mat of deposited solids. Furthermore, the differential increased by not only an increased mat thickness but by any decrease in the permeability of the deposit. Such decreases in permeability (i.e., compression of the once-formed deposit) are the general rule in filtration of algae, etc., as will be seen later by the exponential form of the widely used Boucher's equation in this practice (6).

Differential

The prediction of differential has been achieved in the past by the use of Boucher's equation. The wide acceptance of this relationship suggests that it is applicable for the low concentrations of the algae-like solids at the low flow rates where it was used. Limited tests performed in the previous work (1), however, indicated that the Boucher Index did not predict differential for higher concentrations of combined sewer overflow solids

Table 2

Microstrainer Dynamic Performance Data

Date of Storm	Time Start	Time Stop	Trough Over-flow MGD	Flow-thru MS MGD	Vis-cosity c.p.	Drum sub-mergence %	Drum sub-mergence ft^2(a)
7-1-71	7:55p	8:13p	.94	.52	-	83	7.8
7-19-71	10:33p	11:11p	none	.52	1.150	82	7.7
	11:11p	-	none	.52	1.150	82	7.7
7-29-71	7:10p	7:24p	1.4	.49	1.000	81	7.6
7-30-71	11:15a	12:17p	3.6	.44	1.005	85	8.2
	12:05p	-	3.6	.44 .52	1.005	85	8.2
7-30-71	12:44a	12:50a	1.9	.12	1.005	77	7.3
	12:50a	12:57a	1.9	.52	1.005	77	7.3
7-31-71	12:18a	12:31a	1.9	.47	0.920	83	7.8
8-27-71	3:42a	4:32a	1.4	.52	1.035	82	7.7
	5:50a	6:22a	2.3	.48	1.035	82	7.7
8-27-71	11:35a	12:36p	2.3	.43	1.035	78	7.3
8-28-71	2:35a	2:48a	1.4+	.41	1.020	82	7.7
	3:10a	-	-	-	-	-	-
9-10-71	5:34p	5:52p	4.3	.12	0.955	66	6.2
9-11-71	7:43a	8:05a	2.4	.12	0.992	59	5.5
	9:00a	9:36a	2.3	.12	0.992	59	5.5
	9:50a	10:56a	10.1	.12	0.992	62	5.8
	9:50a	10:00a	10.1	.12	0.992	62	5.8

(a) Total unmasked drum area is 9.4 sq ft of 23 micron aperture Mark 0 screen.

Table 2

Microstrainer Dynamic Performance Data

Differential inch	Drum Speed rpm	Radial Flow Rate gpm/ft^2	Suspended Solids Influent mg/l	Effluent mg/l	Removal %	Sample Procedure(b)
24.0	1.75	46.0	165	65	60	8 min auto.comp.
23.5	6-7	46.5	47	31	34	18 min auto.comp.
23.5	6-7	46.5	317	33	90	hand discrete
23.0	stopped	45.0	27	15	45	16 min auto.comp. hand discrete
24.0	7.00	37.4	115	26	78	30 min hand comp.
24.0	7.00	37.4	72	26	64	hand discrete
21.0	6.00	50.0	39	23	41	auto. discrete
21.0	6.00	50.0	11	28	Incr.	auto. discrete
24.0	7.00	42.0	52	16	69	10 min auto.comp.
23.5	7.00	47.0				50 min auto.comp.
23.5	7.00	43.0	66	1	98	20 min auto. eff. & hand infl.comp. @ 8:00a
21.0	7.00	40.0	29	7	76	64 min hand comp.
23.5	6.70	37.0	effluent sampler failure			
			68	1	98	hand comp.
13.5	3.25	13.5	190	37	80	15 min auto.comp.
8.0	1.70	15.2	90	45	50	15 min auto.comp.
8.0	1.50	15.2	130	15	88	33 min auto.comp.
10.0	2.70	14.5	insufficient sample volume for a composite			
10.0	2.70	14.5	80	36	55	10 min auto.comp.

(b) Refer to Section V for explanation of sample procedure.

Table 2

Microstrainer Dynamic Performance Data

Date of Storm	Time Start	Time Stop	Trough Over-flow MGD	Flow-thru MS MGD	Vis-cosity c.p.	Drum sub-mergence %	Drum sub-mergence ft^2(a)
9-11-71	6:14p	6:37p	2.3	.12	1.005	60	5.6
9-13-71	12:22p	12:30p	.72	.42	1.005	82	7.7
	1:41p	2.35p	4.3	.36	1.005	81	7.6
9-21-71	2:56a	3:08a	1.8	.35	1.005	78	7.4
	3:34a	3:53a	2.3	.44	1.035	81	7.6
10-10-71	6:38a	7:06a	1.2	.36	1.065	78	7.4
	7:06a	7:31a	1.2	.36	1.065	78	7.4
	10:08a	10:40a	.72	.41	1.050	65	6.1
11- 2-71	7:23p	7:41p	1.4	.41	1.020	83	7.8
11-29-71	5:25p	5:47p	10.7	.41	1.308	83	7.8
2-13-72	1:18p	1:44p	.72	.12	1.440	79	7.4

(a) Total unmasked drum area is 9.4 sq ft of 23 micron aperture Mark 0 screen.

Table 2

Microstrainer Dynamic Performance Data

Differential inch	Drum Speed rpm	Radial Flow Rate gpm/ft^2	Suspended Solids Influent mg/l	Effluent mg/l	Removal %	Sample Procedure (b)
9.0	1.90	15.0	96	27	72	20 min auto.comp.
23.0	7.00	38.0	231	6	97	8 min auto.comp.
22.5	6.80	36.5	72	46	36	50 min auto.comp.
21.0	6.10	32.5	33	17	48	8 min auto.discrete
22.5	6.50	41.0	64	32	50	16 min comp.
21.0	5.50	34.0	751	1	99+	28 min auto.comp.
21.0	5.50	34.0	330	4	99	25 min auto.comp.
13.0	4.00	46.5	14	9	36	32 min hand comp.
24.0	7.00	36.4	357	113	69	18 min auto.comp.
24.0	7.00	36.4	115	77	33	22 min auto.comp.
22.0	2.0	16.8	208	106	49	24 min hand comp.

(b) Refer to Section V for explanation of sample procedure.

and another index is now under study. This index shows some promise of predicting performance from known conditions of combined sewer overflow characteristics and environmental factors. Additional work is planned on this new performance which we have termed "the permeability parameter".

The Boucher equation is generally stated as $\ln(H/H_o) = IV$ where V is the volume in gallons passed per sq ft of screen. H is the differential in inches and H_o is the head obtained by extrapolating a plot of ln H vs V back to zero V. I is a Filtrability Index which includes the effect of flow rate used, suspended solids concentration, screen size, particle size and other characteristics of the solids such as bulkiness, etc. In order to illustrate the Boucher equation on Figure 13, the Volume term V is shown as the ordinate (lbs solids applied/sq ft of screen) divided by the suspended solids content of the feed in pounds/gallon (mg/l x 1/120,000). As shown on Figure 13, the determination of Boucher's Filtrability Index requires a laboratory measurement of the differential after passage of at least two increments of water volume using the one water of interest at the one flow rate of interest through a stationary section of the one screen of interest (6). Boucher's Filtrability Index will then permit prediction of differential at other volumes of the same water and the same flow rate. The laboratory investigation is often extended enough to include the "break point" where differential starts to fall. At this break point, the shear forces due to differential reach such a level that much of the previously retained solids are forced through the screen with a sharp reduction of removal efficiency.

The Boucher Filtrability Index, however, cannot be determined directly from data obtained from a rotating drum. Estimates of Boucher's Index from rotating drum data requires the use of a transcendental equation based upon implicit assumptions which do not seem to hold for the suspended solids in combined sewer overflow. The Boucher relation is, therefore, unsuitable for combined sewer overflow and was not used in this phase of the study.

We calculated the permeability parameter of the combined sewer overflow solids in the manner shown on Figure 13. It might be noted here that the true permeability would be a function of the head loss due to passage of a constant flow through a uniform thickness mat of the deposited solids only, corrected for viscosity, and would not include the solids-screen interaction resistance. The elimination of the effect of solids-screen interaction was not possible with our shallow deposits (approximately 0.003 inch). Nor could the effect of changing (increasing) thickness of deposited solids during a rotation cycle be excluded. Also the effect of varying flow rate through an increment of screen area during a

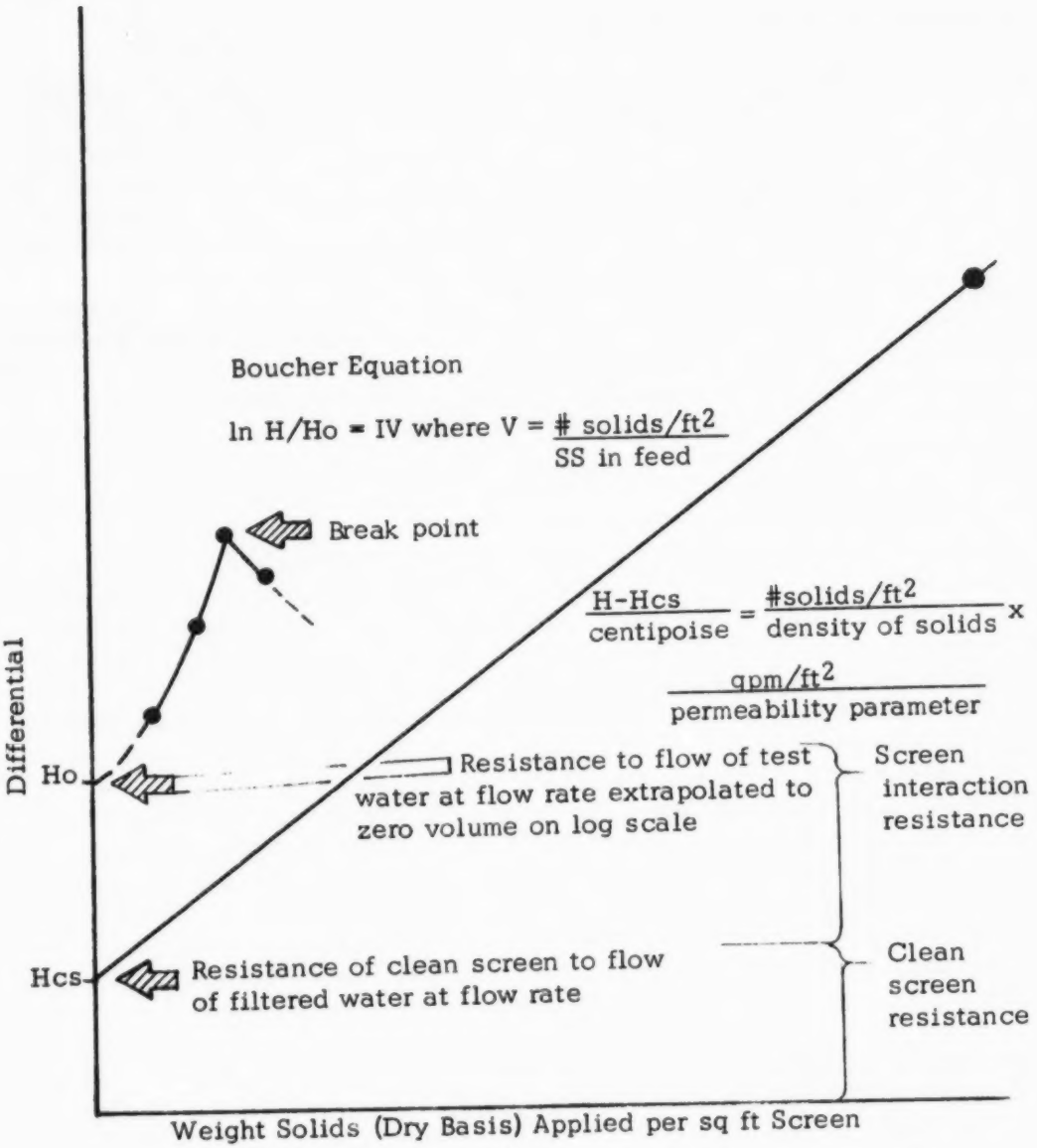
Boucher Equation
ln H/Ho = IV where V = # solids/ft2 / SS in feed
Break point
H-Hcs / centipoise = #solids/ft2 / density of solids x
qpm/ft2 / permeability parameter
Differential
Ho
Resistance to flow of test water at flow rate extrapolated to zero volume on log scale
Screen interaction resistance
Hcs
Resistance of clean screen to flow of filtered water at flow rate
Clean screen resistance
Weight Solids (Dry Basis) Applied per sq ft Screen

Figure 13

Differential vs Applied Solids
(Permeability Parameter & Boucher Index)

submergence cycle could not be excluded. The permeability parameter, then, is the average flow rate in gpm/ft^2 of submerged screen at which the total head loss less the clean screen loss (corrected for viscosity) is one inch of water per inch of maximum or final deposit thickness.

The thickness of the deposit was calculated as follows:

$$\text{Thickness} = \frac{\text{(flow-gpm) (influent SS-mg/l or lb/120,000 gal.)}}{\text{(drum area - ft}^2\text{/rev.) (drum speed - rpm) (bulk density)}} =$$

$$\frac{\text{lb solids applied/ft}^2}{\text{bulk density lb/ft}^3} = \text{ft}$$

The bulk density of the heavier combined sewer overflow solids is expected to range from 60 for clay to 110 lb/ft^3 for sand; other components as leaves, etc., will be lighter. A single measurement of our combined sewer overflow solids showed 85 lb/ft^3 and a bulk density of 60 lb/ft^3 was used in calculating the permeabilities in Table 1. It might be noted that other sorts of wastewater solids are much bulkier. For example: activated sludge solids range from 1 to 0.25 lb/ft^3 settled solids as determined by the Imhoff Cone Analysis. Hydrated alum floc has been reported (7) to be less than 0.2 lb/ft^3 settled solids and algae is probably as light. The bulk density of the retained solids on the screen can be approximated by a measurement of settled solids density similar to Sludge Volume Index (SVI) and is equal to 62.4/SVI. Sludge Volume Index (SVI) is the ratio of the particulate solids content as measured by settling in an Imhoff Cone and reported as ml/l to the weight of suspended solids reported as grams/l. A survey was started late in this phase of the study and will be continued in the next phase to collect information on the bulk density of various solids found in combined sewer overflow. The calculated permeability parameters for each storm are shown on Table 1, Hydrographic Data. The permeability of the deposit, of course, determines the permissible flow rate through the microstrainer for any water with its suspended solids content within the established maximum differential and drum speed limitations. Higher permeability permits higher flow rates and/or operation at lower drum speeds which should promote better solids removal.

The range of permeability calculated was 0.005 to 0.0005, a 10 to 1 range. The permeability parameter in our units of (average gpm/ft^2) x (centipoise) is numerically equivalent to 70 Darcy permeability units as defined by the American Petroleum Institute.

Permeability as an Index of the Character of the Particulate Solids

Permeability by definition combines as a single value the effect of all the properties of the individual particles and of the group of these particles on their resistance to flow. Also, we found empirically that particle retention by the screen increased at higher permeabilities. It can be inferred that many of the particle properties which determine permeability also influence particle retention as well.

The relation of permeability to those particle properties of interest in the liquid-solid separation techniques used in water supply and wastewater treatment forms the basis of the Membrane Refiltration Method of Coagulant Testing (8).

In the next phase of this study, the Membrane Refiltration Method has been selected for evaluating the effect of polymer addition and other pre-treatments of combined sewer overflow prior to a microstrainer.

The characteristics of solids which govern permeability of deposited solids are expected to be important in further analysis of past and present work and in evaluating the effect of polymer addition in future work. It should be informative, then, to consider the art relating to flow through porous and granular beds.

The reported (9) equations for pressure loss through granular media in our range of interest (Reynolds Number 1) indicate that pressure loss is a function of the average particle size, the particle shape, the depth of the media, and the void volume of the solids which in turn is governed primarily by the breadth of the range of particle sizes.

Specifically, then, permeability would be expected to vary directly as (average particle diameter)^2, (1/a shape factor)^2, and a complex function of void volume. The permeability parameter can be expected to be influenced by other factors as well; for example, oil in liquid, etc.

The 10 to 1 range of permeabilities observed in Table 1 could be explained by a 3 to 1 range in average particle size.

Shape factors range from 1.0 for spheres to 1.8 for very angular particles, and thus alone could account for a 3 to 1 range in the permeabilities observed.

The void volume of beds of a narrow size range of any average size particles, if generally spherical shape, is about 40-50%. The broader size range reported in another combined sewer overflow study (10) could reduce the static void volume to approximately 25%. Under flow conditions, the void volume could be further reduced by distortion of individual particles due to compressive forces. Figure 14, however, indicates that our combined sewer overflow solids, during this period, were rigid, and permeability did not decrease with increased compressive force due to increased flow rate, even at the high (24 inches of water) differentials used.

Shown on Figure 14 for reference are typical Permeability-Differential (i.e., Flow Rate) relationships for compressible and non-compressible filter cakes as obtained from Perry's Handbook of Chemical Engineering. A regression analysis showed that the relationship of permeability to flow rate was truly random; that is, the permeability was independent of the radial flow rate as in the case of incompressible solids.

The exponential rise in differential shown on the left-hand curve in Figure 13 with increased deposit thickness of deformable, algae-like solids extensively reported by Boucher (6) indicates decreasing premeability.

Permeability of the solids on the mat increased with increasing concentration of suspended solids in the combined sewer overflow as shown in Figure 15. This relationship, if confirmed by further work, will help define the limiting condition when a microstrainer is applied to high solids waters at high flow rate.

Suspended Solids Removal by a Microstraining Unit

The suspended solids removal performance of the microstrainer is shown in Table 2. The analysis of the first storm of July 1 was made in the field. One or more of the samples during the storm of August 27 at 5:50 a.m. and the storm of August 28 were collected from stagnant contents of the flow-through sample chamber some 5 hours after the storm and, therefore, these three results are not considered in this discussion.

Figure 16 shows that there is no uniform percentage removal nor is there a simple relation between removal efficiency and influent solids concentration.

Figure 16 illustrates the capability of a microstrainer to treat combined sewer overflow; i.e., it is able to reduce combined sewer overflow suspended solids from any level down to generally 40-50 mg/l and lower.

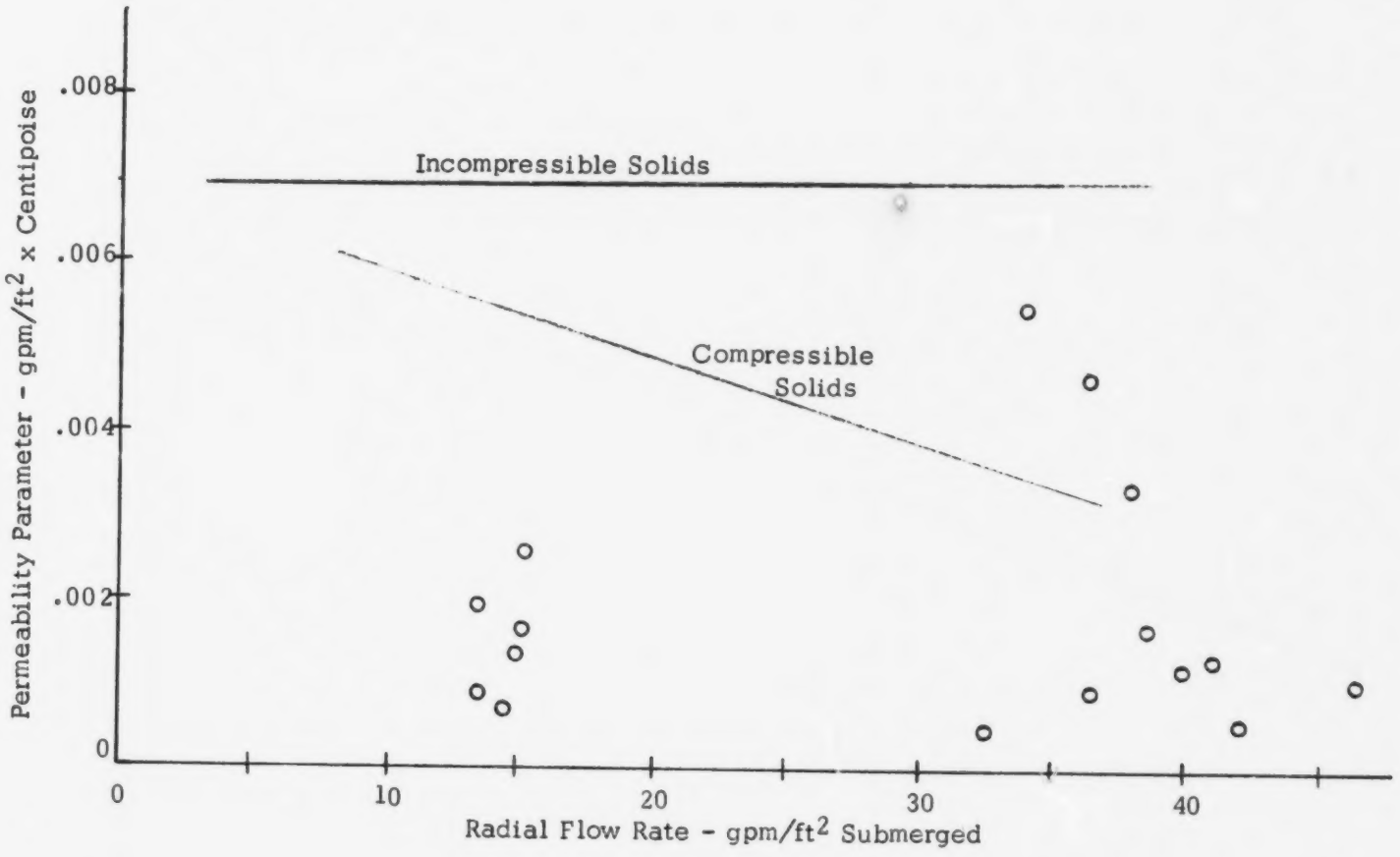

Figure 14

Permeability Parameter vs Radial Flow Rate

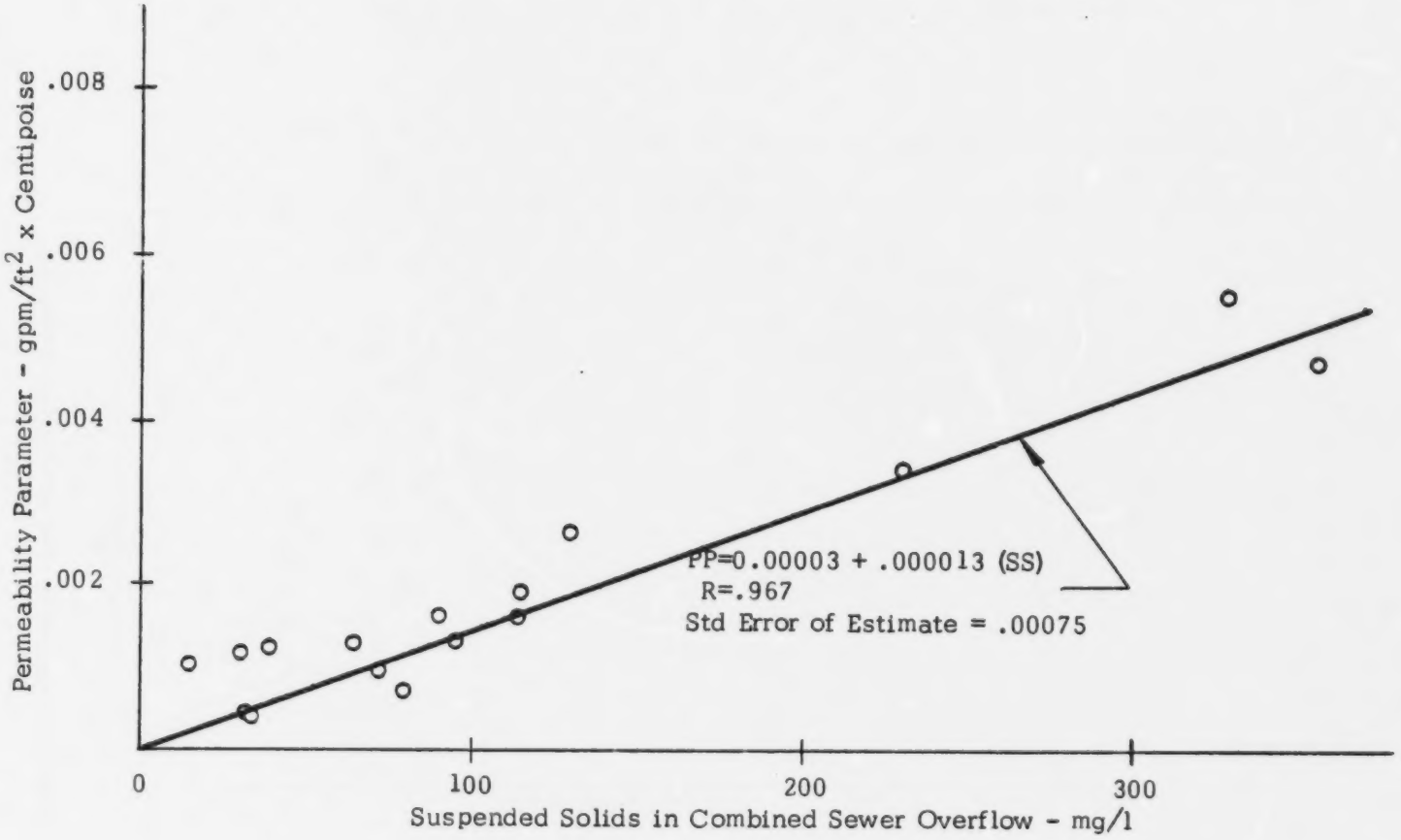

Figure 15

Permeability Parameter vs Influent Suspended Solids

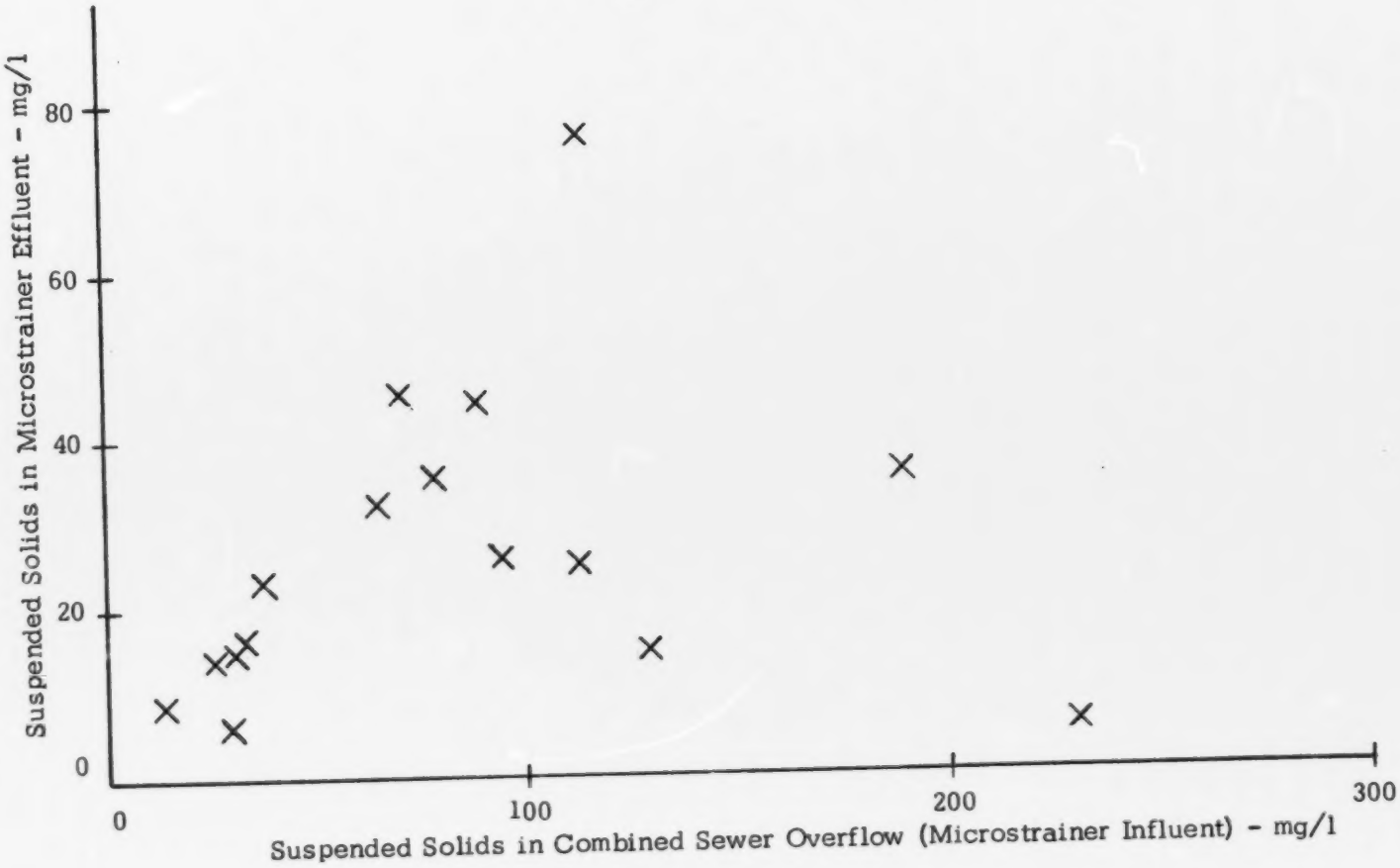

Figure 16

Effluent Suspended Solids vs Influent Suspended Solids

This is shown in another way by Figure 17. This figure shows that the percentage removal as well as the absolute removal increases with high levels of influent suspended solids. Due to the rapidly changing flow rate and character of combined sewer overflows (as compared to domestic wastewaters), the conventionally used percentage removal is an unrealistic parameter for considering both the benefit of any treatment and the Microstraining technique. Fortunately, it is during the intense storms when the potential pollution at the outfall is the greatest that the percentage removal is also the greatest. It seems that a properly sized microstrainer will then limit the pounds of suspended solids per unit time entering the stream from a combined sewer outfall, and this is the ultimate criterion.

Mechanism of Suspended Solids Removal

The actual mechanism of suspended solids retention by a fine screen has been studied by papermakers (11) (12). They find that most of the first increments of solids applied to the screen will leak through, but those solids which are retained form a mat which in turn will reduce the leakage of solids from subsequent increments of applied liquid with its suspended solids.

Figure 18 from Abrams (11) shows the results of a test where successive increments of a suspension of fiber-type solids were applied normal to a stationary, fine, rectangular mesh screen. The initial, or bare screen, retention (shown as tan $\emptyset$) is the dependent variable which determines overall retention for any loading. The initial retention is calculated graphically by an incremental loading plot such as in Figure 18. Also, the initial retention is the dependent variable that Abrams (11) found could be best correlated by independent variables as flow rate, fiber length and screen opening size, etc. As the cumulative mass of fibers reaching the screen is increased, the cumulative percentage retained increases; i.e., the slope of the curve approaches the indicated 45° line asymptotically. In our work, we collected a composite of the entire volume of water approaching and leaving an increment of screen area during a submergence cycle. That is, we determined only a single point (shown as an open circle on Figure 18 on the curve during any one storm. And since the character of our suspended solids apparently varied substantially from storm to storm, as well as during a storm as indicated by the differing permeabilities in Table 1, our points do not fall upon a single curve but rather each point identifies one of a family of curves.

Figure 19 indicates that our combined sewer overflow suspended solids on a moving microscreen behaved similar to paper fibers on a paper forming screen. Thus with combined sewer overflow solids as well, the average removal during a drum rotation cycle is determined generally by the initial

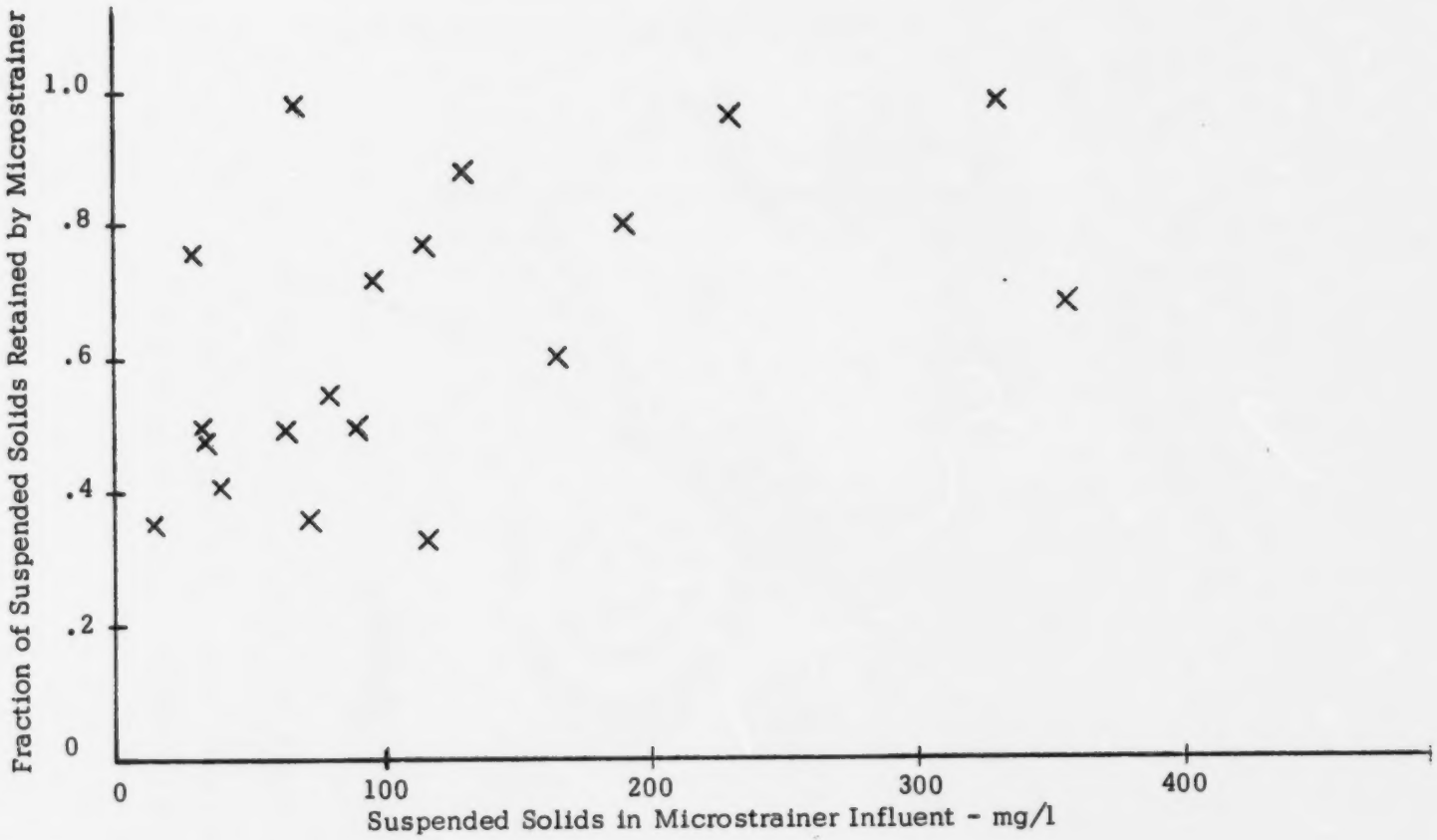

Figure 17

Fraction Solids Retained vs Influent Suspended Solids

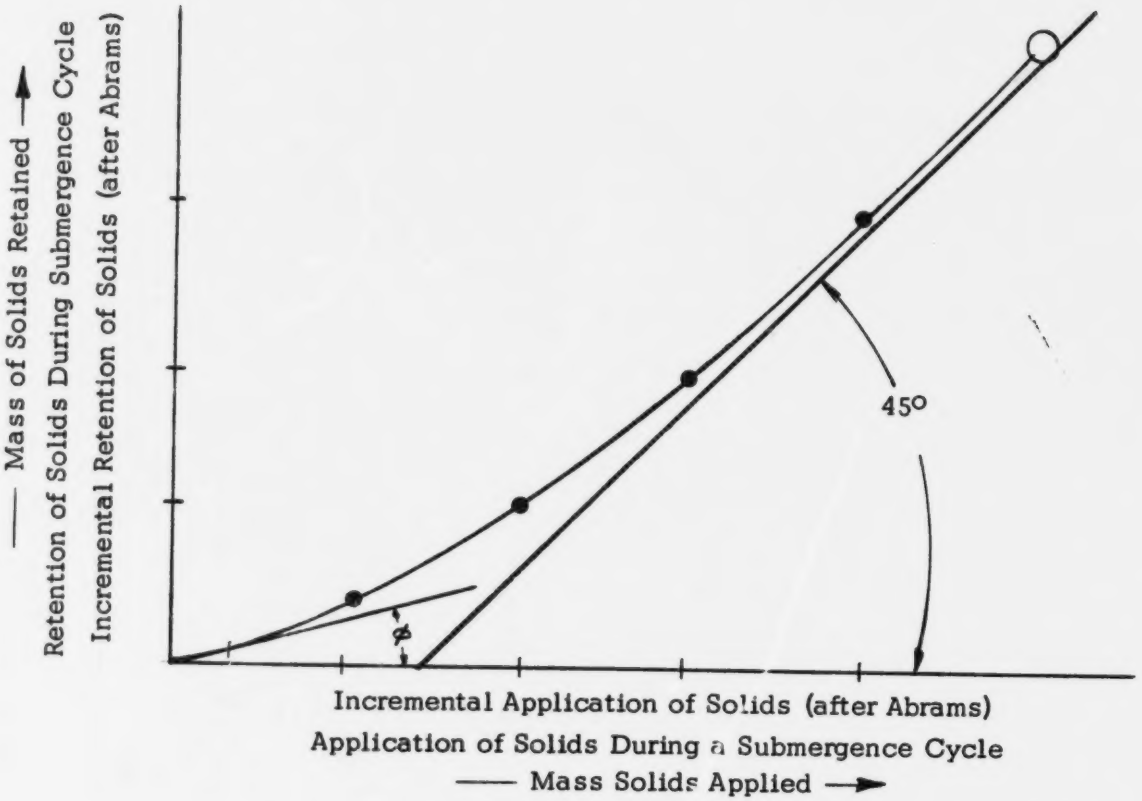
Mass of Solids Retained
Retention of Solids During Submergence Cycle
Incremental Retention of Solids (after Abrams)
45°
Incremental Application of Solids (after Abrams)
Application of Solids During a Submergence Cycle
Mass Solids Applied

Figure 18

Typical Plot of Mass Retained vs Mass Applied

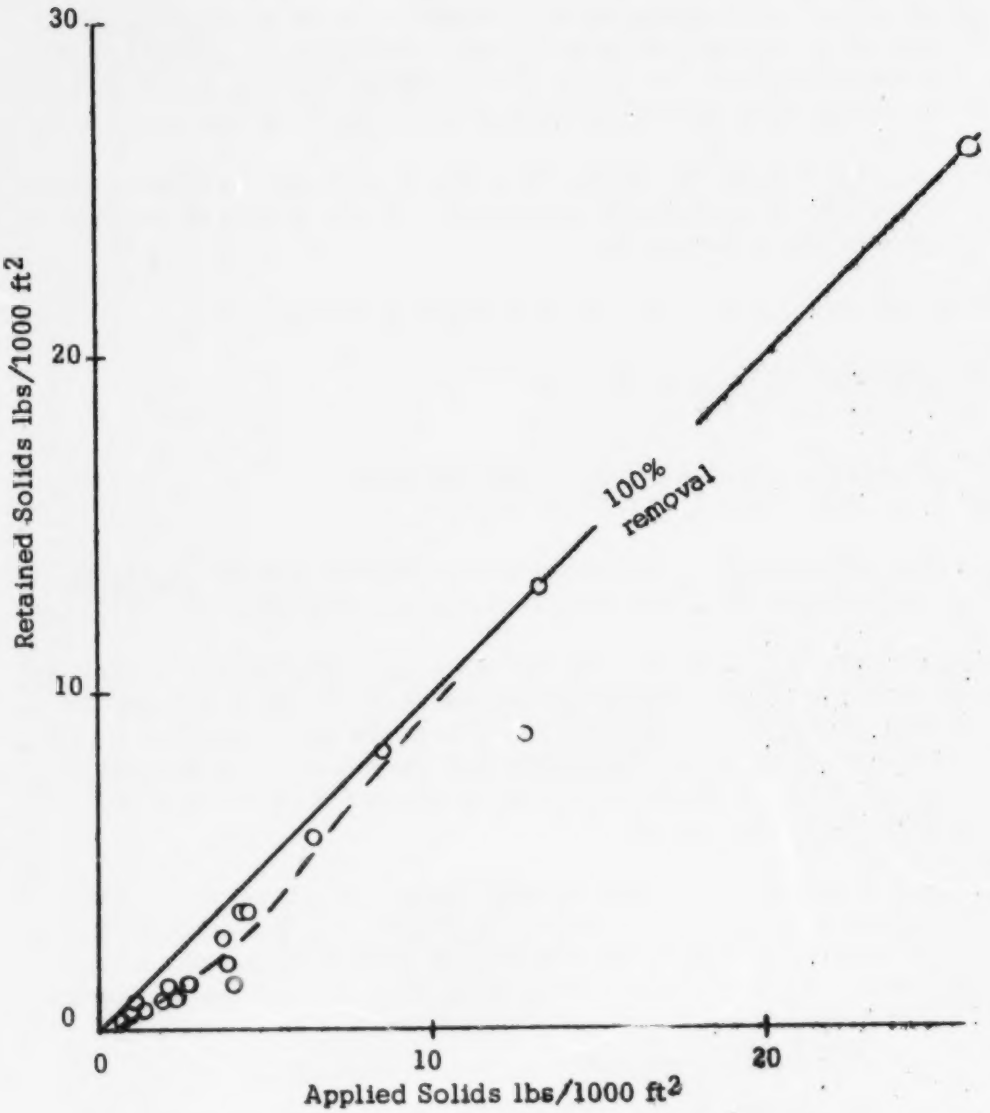

Figure 19

Combined Sewage Solids Retained vs
Combined Sewage Solids Applied

retention. However, from a single point on a removal curve it is possible to only estimate the initial retention. The initial retention parameter is no doubt a function of (a) particle size relative to screen opening size, (b) in the case of rod shaped particles, of the orientation of the approaching particle relative to the direction of flow of the water, (c) the radial velocity of the water relative to the tangential velocity of the screen; i.e., the (tangent of) the angle of approach of the water relative to the plane of the screen, and (d) in the case of the twill mesh microscreen, to the direction of the openings relative to the plane of the screen. Other factors such as surface characteristics of particles may also influence retention.

In any event, the greater the weight of solids applied per sq ft of exposed screen the greater the cumulative percentage removal as shown by the stylized dashed line in Figure 19.

The weight of solids applied per sq ft of exposed screen is:

$$\frac{\text{(flow-through microstrainer-gpm)}}{\text{(drum area - ft}^2\text{/rev)}} \times \frac{\text{(suspended solids in feed-mg/l or lb/120,000 gal.)}}{\text{(drum speed - rev/min)}}$$

This explains the relationship wherein the percentage removal increases with flow rate and as the concentration of suspended solids increase.

It has been previously reported (13) that the lowest drum speed obtainable within the head loss (differential) limitations of the machine yielded the best removal. It might be noted that the slower the drum speed, with all other conditions being equal, the greater the weight of solids applied per sq ft of screen. Thus, we too found, as shown in Figure 19, higher removals at slower drum speeds.

The increase in percentage solids removed caused by higher solids concentration observed at our site is shown by Figure 17. The effect of higher solids concentration is due to the contribution of more solids of any sort being applied to the screen area. Also, it is believed that the additional solids in the higher concentrations at our site are of a more strainable nature; e.g., larger size.

There are indications (6), however, that flow rate per se, in contrast to its desirable contribution to greater weight of solids applied per unit screen area causes per cent removal to decrease. Flow rate causes shear forces which tend to rupture the mat of deposited solids and force it through the screen. This undesirable effect of flow rate is not nearly so pronounced

on the removal of combined sewer overflow solids as with fragile solids like algae or alum floc.

The factors which influence the retention of suspended solids in combined sewer overflows (and other waters) need to be more closely identified. A better understanding of these factors might permit a microstrainer to be applied to a broader range of wastewaters.

In the previous section the flow rate, head loss, and other measurements were reduced to an overall permeability parameter which characterized the solids in relation to their resistance to flow. This flow resistance parameter was empirically found to be loosely related to solids retention performance as well (see Figure 20). Regression analysis showed that the relation was not linear.

It has been shown (9) that the resistance to flow (i.e., 1/permeability) through beds of sand and similar particles is predictable from known particle properties. As will be shown later, these properties can be related to the smallest dimension of the individual particles. Since initial solids retention by a screen is determined by the smallest dimension of a particle, it is not surprising that solids retention is related to permeability.

Filter Head Loss Equation Analogy (9)

$$\left[\begin{matrix}\text{Solids} \\ \text{Retention}\end{matrix}\right] \propto \left[\begin{matrix}\text{Permeability} \\ \text{Parameter}\end{matrix}\right] \propto \left[\frac{(\text{avg particle dia})^2\ (\text{void vol})}{(\text{shape factor})^2\ \ (\text{viscosity})}\right]$$

Obviously, the smaller the average diameter of a group of particles, the smaller the smallest particle will likely be, and the less likely the small particles will be retained initially by a screen.

The shape factor is a property of an individual particle and is numerically 1.0 for spheres and 1.8 for very angular or rod-like particles. An individual particle with a high shape factor has a smaller dimension relative to its nominal diameter than a spherical particle and would be less likely to be retained by a clean screen.

Void volume is a property of a group of particles. It is determined chiefly by the breadth of the range of nominal particle diameters with the narrower size range making for larger void volumes. The shape of the particle has relatively little influence. A low void volume condition would indicate a broad range of individual particle sizes. A broad range of sizes would require that the smallest particle be smaller than the smallest particle in a group of particles of the same average size but of a narrow size range.

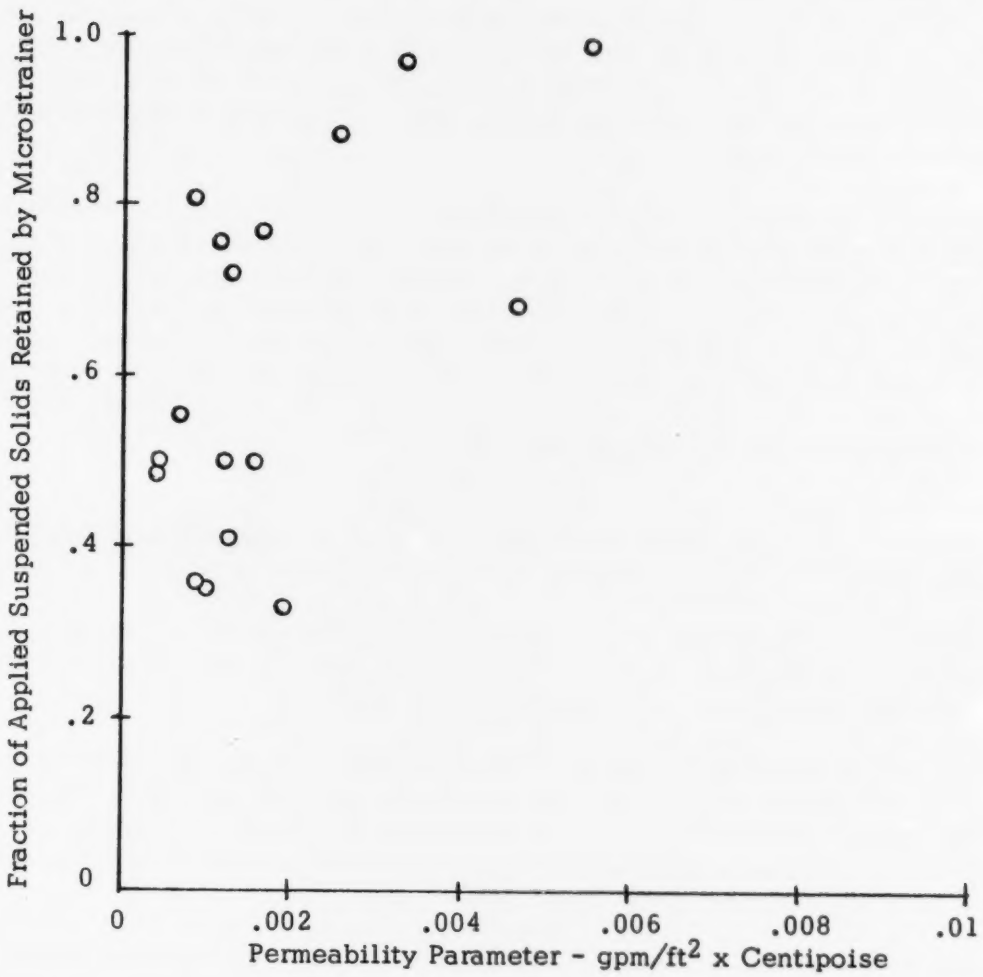

Figure 20

Fraction Solids Retained vs Permeability Parameter

The smaller particles would be less likely to be retained by a clean screen.

In the next phase of this study, an extensive laboratory study will survey the effect of polymer addition and other pretreatments on the removal of solids by a microstrainer. These pretreatments will be evaluated in laboratory glassware by means of a Membrane Refiltration Test (8) which is essentially a permeability measurement. Also, additional storms will be monitored in full scale equipment, as before, without any pretreatment and Membrane Refiltration measurements will be made on the actual stormwater. It is expected that these measurements will reflect full scale performance so that this test can be used to predict microstrainer performance.

Organic Removal by a Microstraining Unit

The organic matter in combined sewer overflow, as in other natural waste streams, is made up of many compounds at varying concentrations. At the arbitrarily selected conditions of the test method (e.g., 5 day incubation period at 20° C in the case of the BOD_5 method), certain fractions of many of these organic compounds will be sensed. In addition to organic matter, other reducing (oxygen demanding) compounds are sensed to some extent by the BOD_5 and COD tests. In particular, ammonia, or organic nitrogen forms which are frequently present in substantial concentrations, exhibit an oxygen demand. The duration and other conditions of the 5 day BOD tests are designed to exclude these nitrogen forms as much as possible. Another measurement of organic matter, TOC (total organic carbon) senses only true organic (carbon containing) compounds and thus furnishes an indication of the total organic material present.

The possibility of residual methanol in the sample bottles rendered many of the earlier samples suspect from the standpoint of interpreting organic removal performance. These storms, and also the storm of August 28 for the reason noted in the Suspended Solids Section, are not considered. Table 3 shows the non-suspect data on organic removal performance of the microstrainer. All of the data, including the suspect data, is shown in Appendix B.

Table 4 summarizes the change in organic content on passage through the microstrainer. As noted in Appendix A, measurements of the standard deviation of the precision of the analytical results at Northeast Laboratory of the City of Philadelphia showed $\pm$ 9% for BOD measurements, $\pm$ 2% for TOC and $\pm$ 5.7% for Suspended Solids measurements. The 13th Edition of Standard Methods indicate deviations of $\pm$ 6.5% for COD, $\pm$ 6.5% for Suspended Volatile Solids and $\pm$ 5% for Dissolved Volatile Solids. In Table 4 all changes of less than the deviation noted in the table are listed as instances of no change.

Table 3

Organic Removal Performance

Date of Storm	Radial Flow Rate[d]	S.S. Infl mg/l	SVS In mg/l	SVS Out mg/l	SVS Removal %	DVS In mg/l	DVS Out mg/l	DVS Removal %
7-29-71	45.0	27	22	12	45	34	67	Incr
	45.0	27	-	-	-	-	-	-
	45.0	27	-	-	-	-	-	-
7-30-71	37.4	115	29	4	87	18	16	15
	37.4	115	-	-	-	-	-	-
	37.4	72	18	7	69	-	-	-
8-27-71	40.0	29	16	6	63	10	17	Incr
	40.0	29	-	-	-	-	-	-
	40.0	29	-	-	-	-	-	-
9-13-71	36.5	72	24	17	29	ND	ND	-
	36.5	72	-	-	-	-	-	-
10-10-71	34.0	330	81	3	96	ND	ND	-
	34.0	330	-	-	-	-	-	-
	34.0	330	-	-	-	28	39	Incr
	46.5	14	7	4	43	ND	ND	-
	46.5	14	-	-	-	-	-	-
	46.5	14	-	-	-	11	16	Incr
11-2-71	36.4	357	76	48	36	30	32	Incr
	36.4	357	-	-	-	ND	ND	-
11-29-71	36.4	115	31	23	26	ND	ND	-
	36.4	115	-	-	-	-	-	-
	36.4	115	-	-	-	17	18	Incr
2-13-72	16.8	208	49	30	39	52	19	63
	16.8	208	-	-	-	-	-	-
	16.8	208	-	-	-	-	-	-

ND - Not Determined

[d] gpm/ft^2

Table 3

Organic Removal Performance

5 Day BOD			COD			TOC		
In	Out	Removal	In	Out	Removal	In	Out	Removal
mg/l		%	mg/l		%	mg/l		%
30	30	no change	79	68	14.0	ND	ND	-
28	28	no change	89	108	Incr	39	29	25.0[a]
20	ND	-	-	-	-	-	-	- [c]
260+	252+	-	2350	6270	Incr	ND	ND	-
348	256+	-	2160	10200	Incr	864	3154	Incr [a]
ND	ND	-	-	-	-	-	-	- [b]
880	550	37.5	3760	1830	51.5	ND	ND	-
940	440	53.0	1410	990	30.0	409	224	45.0[a]
1180	540	54.0	-	-	-	-	-	- [c]
1730	1740	Incr	3930	3390	13.5	ND	ND	-
2150	1250	42.0	4460	3570	20.0	1065	810	24.0[a]
129	112	13.2	256	216	15.6	ND	ND	-
112	98	12.5	280	248	11.4	78	54	30.8[a]
106	124	Incr	-	-	-	-	-	- [c]
8	7	12.5	32	200	Incr	ND	ND	-
7	7	no change	384	416	Incr	10	10	no change[a]
7	7	no change	-	-	-	-	-	- [c]
55	26	52.5	230	170	26.0	77	38	51.0
69	34	51.0	250	180	28.0	ND	ND	- [a]
12.5	26	Incr	117	117	no change	ND	ND	-
13.5	27.5	Incr	110	61	45.0	23	22	4.0[a]
-	-	-	-	-	-	-	-	- [c]
6	16	Incr	81	76	6.0	ND	ND	-
5	15	Incr	68	41	40.0	24	21	12.5[a]
2	8	Incr	-	-	-	-	-	- [c]

Sample Procedure

[a] macerated composite
[b] discrete
[c] filtered

ND - Not Determined

Table 4

Effect of Microstraining on Organic Matter as Indicated by Several Test Methods

Change in Content of Organic Matter by Microstraining	Total Organic					Dissolved Organic		Solid Organic
	BOD_5		COD		TOC	DVS	BOD_5 filtered	SVS
	unmacer-ated	macer-ated	unmacer-ated	macer-ated	macer-ated	macer-ated	macer-ated	unmacer-ated
No Changes								
Number of Instances	5	3	2	2	2	3	2	0
Increase								
Number of Instances	2	2	2	2	1	4	1	0
Average of Increases mg/l	12	12	2050	4010	2300	14	6	0
Average of Increases %	138	126	345	145	165	63	300	0
Decrease								
Number of Instances	2	4	5	5	6	1	1	10
Average of Decreases mg/l	180	380	516	290	86	33	640	20
Average of Decreases %	45	46	24	33	32	63	54	53
Deviation of Precision of Test	± 9%*		± 6.5%**		±2%*	± 5%**	± 9%*	± 6.5%**

All changes in concentration upon passage through microstrainer of less than the deviation are considered no change.

*Deviation of precision determined this work.
**Deviation of precision predicted by Standard Methods.

With this assumption, the data in Table 4 indicates rather clearly that any changes in total organic matter were in the direction of a significant reduction of some 25-45%. The reductions in total organic matter are some 200 mg/l BOD_5, 400 mg/l COD and 100 mg/l TOC in magnitude. The extraordinary high average increases shown are heavily influenced by the single storm of 7/30/71.

The change in suspended (solid) organic matter on passage through the microstrainer was in the direction of a decrease in every case. As shown in Table 4, the decreases in Suspended Volatile Solids (SVS) averaged 21 mg/l or 51%. Also, as shown in Figure 21, the reduction in SVS paralleled the reduction in Total Suspended Solids (SS). The basis for the SVS test is so dissimilar from the bases of the BOD, COD and TOC tests that no absolute comparison can be made. However, the magnitude of the total organic reduction, as indicated by all tests except the Ultimate BOD, is 5 to 20 times the suspended organic reduction as measured by the SVS. The magnitude of this difference suggests, we believe erroneously, that some forms of organic matter other than suspended organic are removed by the microstrainer.

The change in dissolved organic matter on passage through the microstrainer is shown in Table 4 by the Dissolved Volatile Solids (DVS) and Filtered BOD_5 tests.

No good explanation is available for the four instances of increase in soluble organic matter indicated by the DVS test. The increases were, however, small in magnitude, averaging 14 mg/l, and are perhaps due to poorer than anticipated precision of the DVS test.

The filtered BOD_5 data shows two instances of no change, one very small increase, and one instance of a large decrease.

It seems from the above that there is little if any change in the soluble organic matter on passage through the microstrainer.

The ultimate BOD and rate constant data from non-suspect, unmacerated and macerated samples are shown on Table 5. These data follow the same pattern as that previously discussed for BOD_5, COD, and TOC.

One storm showed practically no change (0 to 2 mg/l) on both unmacerated and macerated samples. Two storms showed small increases (9 to 16 mg/l) on both unmacerated and macerated samples. The other two storms showed decreases (32 to 80 mg/l).

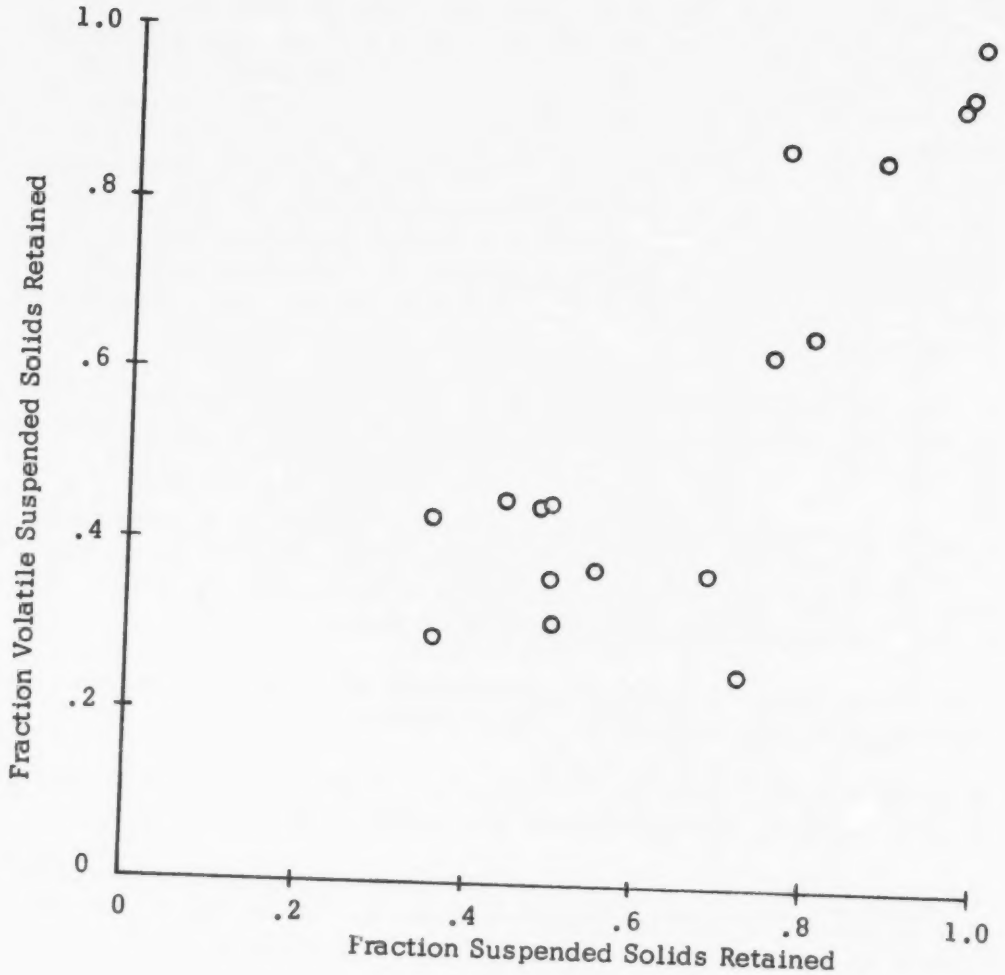

Figure 21

Fraction Volatile Suspended Solids Retained vs
Fraction of Suspended Solids Retained

Table 5

Ultimate BOD and Oxidation Rate Studies

	UNMACERATED SAMPLE						MACERATED SAMPLE					
	Ultimate BOD			Rate Change			Ultimate BOD			Rate Change		
	In	Out	Change	In	Out	Change	In	Out	Change	In	Out	Change
	mg/l		%	mg/l-day		%	mg/l		%	mg/l-day		%
7/29	220	140	- 36%	-	-	-	220	150	-31%	-	-	-
10/10	12	10	- 15%	.290	.260	- 11%	17	17	0%	.800	.730	- 9%
11/2	65	33	- 49%	.327	.234	- 28%	86	51	-43%	.237	.176	-26%
11/29	14	30	+100%	.433	.390	- 10%	18	30	+75%	.322	.447	+14%
2/13	14	27	+ 93%	.090	.195	+116%	16	25	+56%	.092	.145	+58%

The sample maceration technique is strongly recommended for any further work on combined sewer overflow. On the storms showing a decrease in organic matter across the microstrainer as measured by BOD_5 and COD the decrease shown was always higher with macerated samples. This is attributed to the character of the suspended and colloidal organic matter in combined sewer overflow and the apparent effect of maceration of the influent sample.

It is believed that maceration renders the organic matter more available to the analytical procedure.

It has been theorized that the same affect can be produced across a microstrainer and is due to the very intense shearing (estimated G = 100,000+ sec^{-1}) for a fraction of a second (.001 sec) caused by passage of water through the mat of deposited solids on the microscreen. For comparison, in the laboratory maceration process, the sample is subjected to intense agitation (approximately 60,000 sec^{-1}) for 60 seconds.

The possible magnitude of this increase in availability of suspended organic material to oxidation due to shearing is indicated by the difference in measurements between the laboratory macerated sample and the untreated sample. The laboratory macerated influent samples averaged 17% more BOD_5 than the untreated influent samples and generally the same or perhaps 5% more COD. See Figures 22 and 23. The actual extent of the effect of the Microstraining unit's operation on availability is indicated by the fact that macerated effluent samples averaged only 14% more BOD_5 than the unmacerated effluent sample. That is, the microstrainer operation appeared to make available (to the BOD_5 test procedure) some 18% of the unavailable suspended solids organic matter in the combined sewer overflow. The effect of maceration of microstrainer effluent samples prior to COD analysis is uncertain.

The rate constant is also directly related to the availability of organic matter to an oxygen supply. This availability is increased by higher particle surface to volume ratio. A tendency was noted for increased rate constant with more severe straining conditions (i.e., higher differential, higher radial flows, etc.). Similarly, the ultimate demand exhibited a pattern of decrease with more efficient removal of SVS.

Again, these observations did not hold in all cases. However, on the average, the effect of a Microstraining filter under high differential, high radial flow conditions was towards increased rate and decreased ultimate BOD values.

Laboratory maceration has the same effect on BOD rate constant and

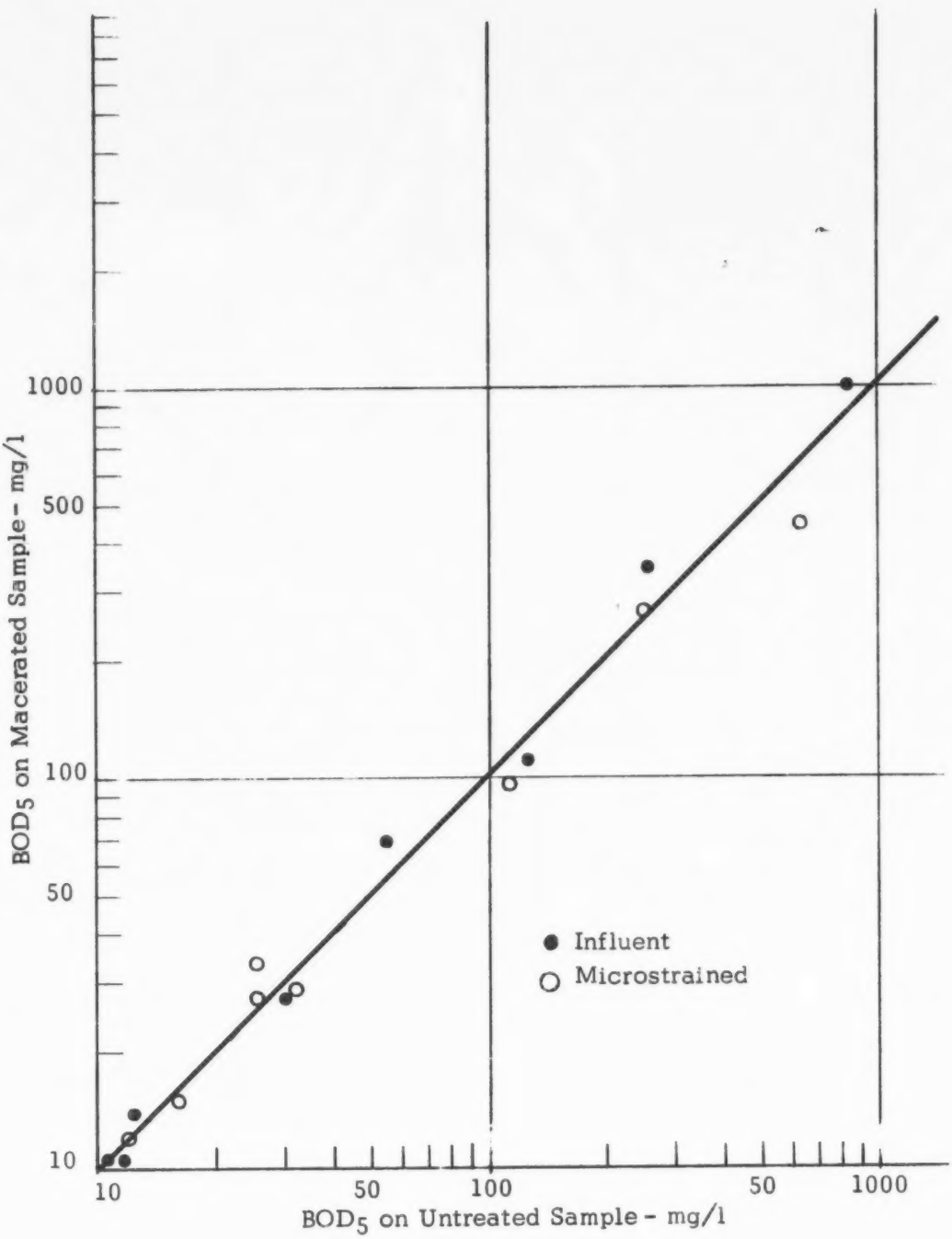

Figure 22

BOD_5 vs BOD_5 on Macerated Sample

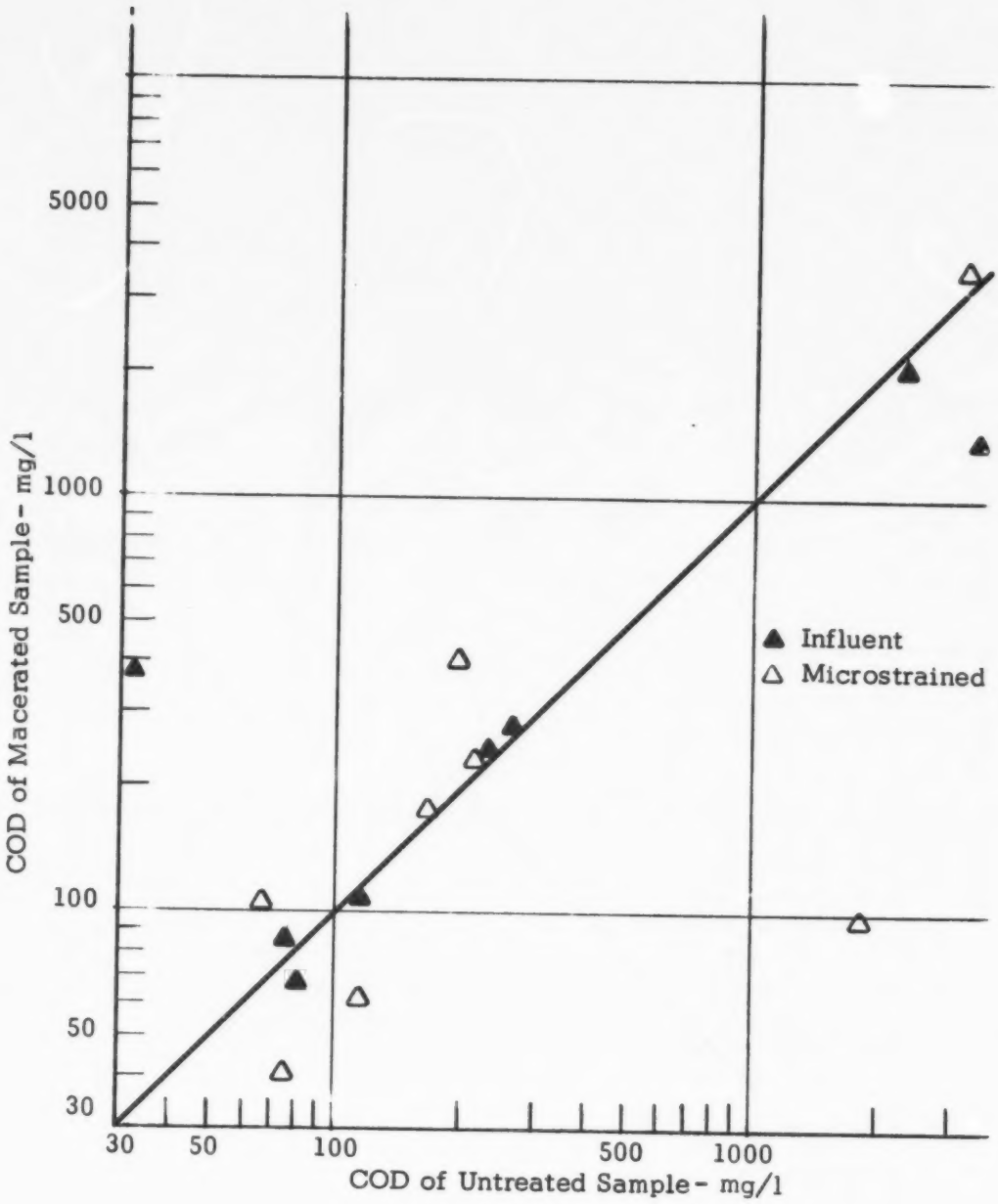

Figure 23

COD vs COD on Macerated Sample

ultimate value. The effect is more pronounced on influent samples than on the effluent due to the maceration a sample is given by the microstrainer process. The fact that there were differences in all cases supports the recommendation that samples be macerated prior to analysis.

In general it seems that the sample maceration pretreatment and the more reliable, chemically determined, COD and TOC analytical methods yielded the most consistent results. The rapidly changing and variable average level of contaminants, together with the inherent time lag in the method, make the BOD and related biological tests inappropriate for combined sewer overflow service.

The maceration technique, as used here, has been useful and its use is strongly recommended for all combined sewer overflow organic analyses as well as those on other similar wastewaters.

The need for refinement of this technique, particularly when used prior to bacteria counts, will be discussed later.

Coliform Removal by a Microstraining Unit

The coliform data which are not suspect because of possible methanol contamination are shown in Tables 6 and 7. The complete data including suspect samples are shown in Appendix C and Appendix D.

Coliform counts were performed by the 12th Edition of Standard Methods. In this method there are several difficulties. First, the coliform count of combined sewer overflows varies unpredictably, as shown on Tables 6 and 7 over several orders of magnitude. It is difficult, then, to preselect the proper sample dilution ratio in order to arrive at a countable concentration of cells, which is a narrow range. Thus, as shown by footnotes on Tables 6 and 7, the majority of the counts were made on dilutions well outside the ideal counting concentration range. Second, the diluted sample is passed through a very fine filter which entraps all sorts of bacteria in addition to the type of interest; i.e., fecal coliform. The membrane filter with entrapped bacteria is then placed in an incubating broth whose composition as well as the incubation temperature and duration are selected to foster the growth of only the single type of bacteria of interest. The actual count made is of the colonies produced by each surviving cell. Thus, a clump of many cells in the original sample would produce a single colony and would be counted as a single cell. To reduce the possibility of clumping and of cells isolated from nutrient solution by a solids barrier, an aliquot of the sample was subjected to intense maceration in a Waring blender. Counts were made upon both the macerated and the untreated samples.

Table 6

Total Coliform Removal by Microstraining

Date of Storm	Radial Flow Rate gpm/ft^2	S.S. Influent mg/l	Temp °F	pH (1)	Across Microstrainer		
					Influent	Effluent	Removal
					Cells/100 ml x 10^6		%
7-29-71	45.0	27	69	7.1	2.800	0.001(3)	99+
7-30-71	37.4	115	68	6.9	3.300	0.180	94.5(4)
					0.900	0.130	85.6
8-27-71	40.0	29	68	7.1	0.870	0.880	no change (4)
					0.850	0.092	89.2
9-13-71	36.5	72	68	7.3	0.730	0.550	24.7(4)
				7.2	0.530	0.400	24.5
10-10-71	34.0	330	64	6.7	0.780	4.200	Incr(4) (540%)
				7.7	1.400(2)	0.004(2)	99+
	46.5	14	65	7.5	2.100	0.008(2)	99+
				7.5	0.900	0.050	94.4
11-2-71	36.4	357	67	7.3	1.100(2)	1.360(2)	Incr(4) (123%)
				7.4	0.320	0.800	Incr (250%)
11-29-71	36.4	115	50	7.3	0.760	0.680	10.0(4)
					0.570	0.780	Incr (134%)
2-13-72	16.8	208	45	7.0	0.340	0.850	Incr(4) (250%)
				7.5	0.005	0.070	Incr (1400%)
					0.004	0.036(2)	Incr (900%) (4)

(1) pH of filter influent
(2) Estimated count based on non-ideal colony count
(3) Zero plate count based on 0.1 ml sample
(4) Values are results on analyses of another aliquot of same composite after intense maceration

Table 7

Fecal Coliform Removal by Microstraining

Date of Storm	Radial Flow Rate gpm/ft^2	S.S. Influent mg/l	Temp °F	pH (1)	Across Microstrainer		
					Influent Cells/100 ml x 10^6	Effluent	Removal %
7-29-71	45.0	27	69	7.1	0.0740	.0010(4)	98.7
					0.0610	.0020(2)	97.0(5)
					0.0010	.0001(3)	90.0
7-30-71	37.4	115	68	6.9	0.0010	.0008	20.0(5)
8-27-71	40.0	29	68	7.1	0.0023	.0030(2)	Incr (130%)
				7.3	0.0220	.0060(2)	73.0(5)
9-13-71	36.5	72	68	7.2	0.1330(2)	.0190	85.7
				6.7	0.0018(2)	.0470	Incr(5) (2600%)(2)
10-10-71	34.0	330	64	7.5	0.0270	.0001(3)	99.6
				7.5	9.2000	.0001(3)	99+(5)
	46.5	14	65	6.9	0.0170	.0200	Incr (118%)
				7.3	0.0530	.0450	15.0(5)
11-2-71	36.4	357	67	7.4	0.0044	.0270	Incr (670%)
					0.0088	.0150	Incr (170%) (5)
11-29-71	36.4	115	50	7.3	0.0470	.0520	Incr (110%)
					0.0480	.0780	Incr (164%) (5)
2-13-72	16.8	208	45	7.0	0.0600	.0640	Incr (107%)
					0.0140	.0150	Incr (107%) (5)

(1) pH of filtered influent
(2) Estimated count based on non-ideal colony count
(3) Zero plate count based on 1 ml sample
(4) Zero plate count based on 0.1 ml sample
(5) Values are results on analyses on another aliquot of same composite after intense maceration

As can be seen, either clumping and/or occlusion of cells in solids could have occurred (Figures 24 and 25). The macerated sample compared to the untreated sample showed higher counts in 25 cases, a lower count in 10 cases, and the same count in one case. The increase ranged from 10 to 100%.

The effect of a Microstraining unit is shown by the fecal coliform data (neglecting changes of 0.001 million cells/100 ml and changes of less than 10%) by the standard count: three instances of no change, three increases, and three decreases. Similarly, comparison of the microstrainer influent and effluent samples after maceration showed: two instances of no change, three increases, and four decreases.

On the same basis, the total coliform data showed by the standard count: no instances of no change, three increases, and six decreases. Similarly, the macerated samples showed: two instances of no change, four increases, and three decreases.

The conclusion regarding coliform removal drawn from the data is that there is no significant reduction across the microstrainer.

Disinfection of Combined Sewer Overflows by Chlorination

The extent of disinfection by chlorination has been shown to be a function of chlorine residual, pH, temperature and contact time (14). The first three of these parameters pertain to the chemical form and activity at which the disinfecting agent exists. Contact time determines the extent of contact made possible by the equipment in use.

The specific effect of some of the individual parameters, especially contact time and residual chlorine, has been shown in many studies. Until recently, however, there has been little information of the effect of mixing intensity. Collins, et al (15) and Kruse et al (16) have clearly shown the magnitude of the influence of mixing intensity by the comparison of bacterial kills with two mixing intensities. In Collins' work using identical conditions - 5 mg/l chlorine residual, 1 minute contact time, pH 7, and 15° C temperature - a tubular reactor yielded a survival ratio of 6×10^{-4} while a mechanically stirred reactor yielded a ratio of 1×10^{-1}. In Kruse's work, the effect of a flash mixing compared with gentle stirring at 5 mg/l chlorine dosages and at 1 minute total contact at pH 7 yielded survival ratios of 1×10^{-2} and 5×10^{-1} respectively. Also, it might be noted here that a conventional chlorine contact unit under similar conditions might yield a ratio of 9×10^{-1} if, indeed, the chlorine containing chemicals were dispersed at all in 1 minute.

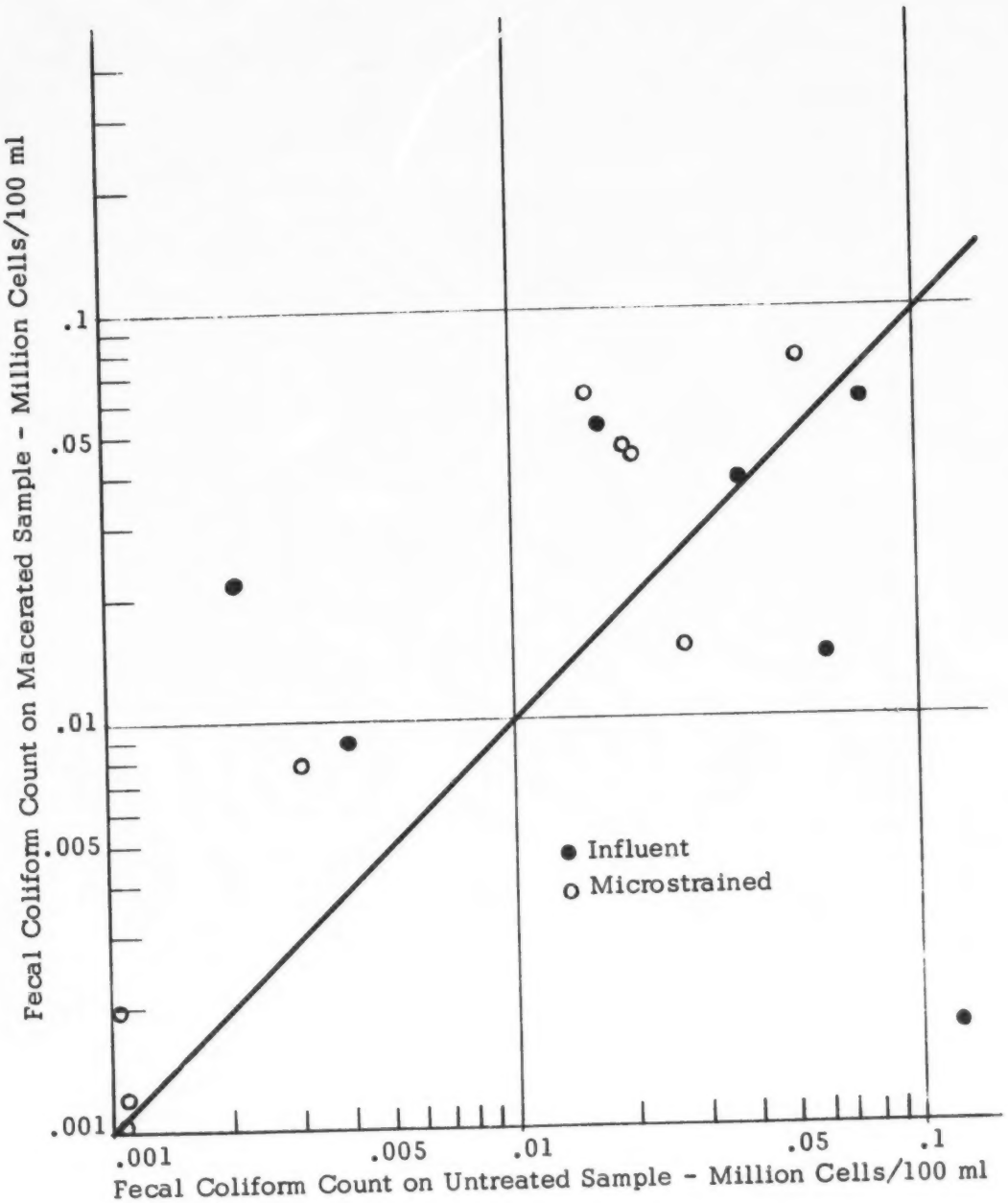

Figure 24
Fecal Coliform Count on Macerated Sample vs
Fecal Coliform Count on Untreated Sample

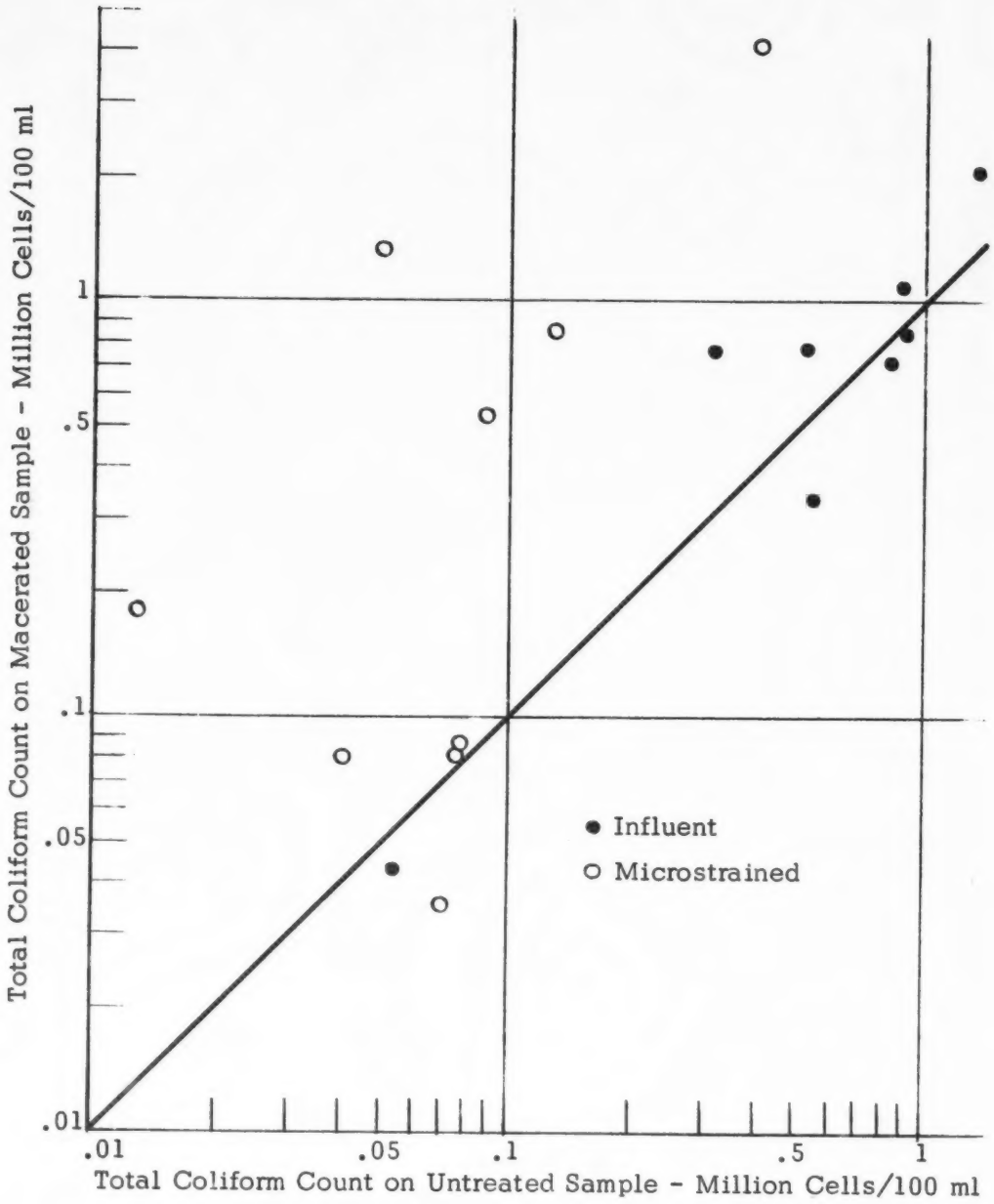

Figure 25

Total Coliform Count on Macerated Sample vs
Total Coliform Count on Untreated Sample

Conventional chlorination is typically carried out in units with residence times of 15 to 30 minutes with a residual of 1/2 to 2 mg/l available chlorine. Such equipment requires high capital costs in land and concrete especially when provided for the very high flow rates of combined sewer overflow service. They are often inefficient in providing contact due to poor mixing and the likelihood of short circuiting. Operating costs are often magnified because higher chemical dosages are required to bring bacterial counts within acceptable standards. For the purpose of disinfecting combined sewer overflow, the provision of the customary 30 minutes contact time is especially inconsistent in combination with a microstrainer having 2 minute residence time.

Also as pointed out by Field (21), the temperature of combined sewer overflow will likely be considerably hotter or colder than the influent to a sewage plant. The lower temperature will reduce the rate of reaction and require much higher chlorine residuals, and/or much longer equivalent contact time. Substitution of more mixing intensity or more dosage will be especially important in low temperature conditions.

The development of design criteria for short duration chlorine contact processes is without precedent. However, in the related field of water flocculation, where a high degree of contact must be obtained in short mixing times, a design procedure has been developed which is gaining acceptance in the industry (16) (17). This relationship was used to design the 4 minute contact chambers.

An equation for the velocity gradient, "G", for mixing intensities in a flowing open channel was developed (18).

$$G = \frac{1730}{\sqrt{\mu}} \sqrt{VS}$$

Where μ = viscosity, centipoise
V = velocity, ft/sec
S = slope, $\frac{\text{ft head loss}}{\text{ft travel}}$
G = velocity gradient, sec^{-1}

Camp and Stein (19) showed that velocity gradient is a measure of mixing intensity and that it is in fact a measure of the number of opportunities for particle collisions per unit time per unit volume.

Velocity gradient has units of sec^{-1}. Then G x Volume x Time yields a design number representing the total number of expected particle collisions. For equal collisions in a batch process where volume is held constant,

$$G_1V_1T_1 = G_2V_2T_2, \; V_1 = V_2 \text{ and, therefore,}$$
$$G_1T_1 = G_2T_2$$

It is seen here that contact (batch) time may be reduced to the extent to which mixing intensity is increased, and the same number of particle collisions is achieved.

In the continuous (flow-through) process where residence time is generally the controlled factor, equal collisions occur when:

$$G_1V_1T_1 = G_2V_2T_2,\ T_1 = T_2 \text{ and, thus,}$$
$$G_1V_1 = G_2V_2$$

Of course, contact volume is directly related to residence time for any flow rate and, for convenience, the duration to which a flow is subjected to disinfection is reported as a residence time. Again, residence time (and thus residence volume) may be reduced as long as the velocity gradient is proportionally increased.

Note in the defining equation for the velocity gradient that it is related directly to the square roots of both velocity and head loss per unit length of travel. Head loss is, in turn, a function of velocity. Therefore, by increasing velocity and inducing a greater head loss per unit length through the use of corrugated baffles and other techniques, the magnitude of the velocity gradient may be increased to offset a reduced contact chamber volume.

It should be noted here that all size chambers, not only high velocity gradient chambers, should be designed to avoid short circuiting to the maximum possible extent. That is "backmixing" must be avoided and "plug flow" attained. The baffle arrangements which will promote plug flow are ineffective for gross mixing. Thus an effective pre-mix with disinfection chemicals is required before the water enters the actual contact chamber.

Table 8 illustrates the relative efficiencies of the four disinfection instruments discussed thus far. The velocity gradient in the tube reactor used by Collins et al (15) has been calculated rather exactly from their reported velocity, tube diameter and temperature (20). G for their batch reactor has been estimated from scanty information (20). Actual velocity and slope measurements have been taken to calculate the gradient for both a local conventional basin and the unit designed by the authors for disinfection of combined sewer overflows. Insufficient information is given regarding the equipment to estimate the velocity gradient in the work of Kruse et al (16). The results of a limited number of studies with our corrugated baffle chambers are shown for comparison in Table 8.

Table 8

Comparison of Disinfection Devices

Fraction of Organisms Surviving Dimensionless	Velocity Gradient Sec^{-1}	Contact Time Sec	GT Dimensionless	Chlorine Residual mg/l	Type Chamber
6×10^{-4}	6800	60	4×10^{5}	5	tube (15)
1×10^{-1}	400	60	2.4×10^{4}	5	stirred batch (15)
3×10^{-4}*	40	240	1×10^{4}	3	corrugated baffles
1×10^{-3}*	40	120	4.8×10^{3}	3	corrugated baffles
1×10^{-2} est.	6	1800	1.1×10^{4}	-	conventional

*Based on limited tests.

The high intensity mixing, high rate contact chamber used is shown in Figure 2. Chlorine in the form of sodium hypochlorite solutions of predetermined strength was initially fed directly into a 15 second residence mixing chamber by a variable capacity Lapp pump. Midway in the disinfection study program a homemade column filled with an inorganic cationic ion exchanger in the hydrogen form was added between the pump discharge and introduction to the chamber inlet stream. Sodium hypochlorite solutions are naturally alkaline (pH 9-10) in which state the less active hypochlorite ion, OCl^-, exists. Passage through the ion exchangers lowers pH to the 6-7 range, coverting hypochlorite ion to hypochlorous acid (HOCl), the more powerful oxidizing agent.

By varying both sodium hypochlorite feed strength and pump delivery rate into a constant 20 gpm water feed, dosages between 0 and 15 mg/l available chlorine may be added. Not shown in the figure are product drawoff lines representing 1, 2, and 3 minutes of contact time (or contact volume). A dosage-time study was then simply a matter of setting for a single dosage and sampling from the influent mixing chamber (at 15 seconds residence), product drawoff lines, and effluent.

Two units are presently installed at the project site. Microstrainer influent and effluent waters were treated simultaneously in order to determine any differences in ease of disinfection brought about by the removal of solids and other effects of the Microstraining unit. For this study, the chlorine residual was matched at the 3 minute product drawoff points for influent and effluent, compensating for the greater chlorine demand exhibited by the influent combined sewer overflow.

Three studies have been made during three separate storms in the test period, each at a velocity gradient of $G = 40\ sec^{-1}$.

Figures 26 through 31 show that the Microstraining pretreatment did not increase the disinfection rate in the high rate chambers used in this study as we had expected it would (1). The effect of Microstraining on "ease of disinfection" in conventional rate equipment has not been determined.

The first of these was made on the storm of 11:30 a.m., August 27, 1971, using 15 mg/l chlorine dosage, and results are depicted graphically in Figure 26 and Figure 27. The 3 minute residuals for this test were each 12 mg/l.

A second study was made during the storm of 1:40 p.m., September 13, 1971, using 10 mg/l chlorine dosages. The results are illustrated in Figure 28 and Figure 29. Three minute residuals on this study matched at 7 mg/l each.

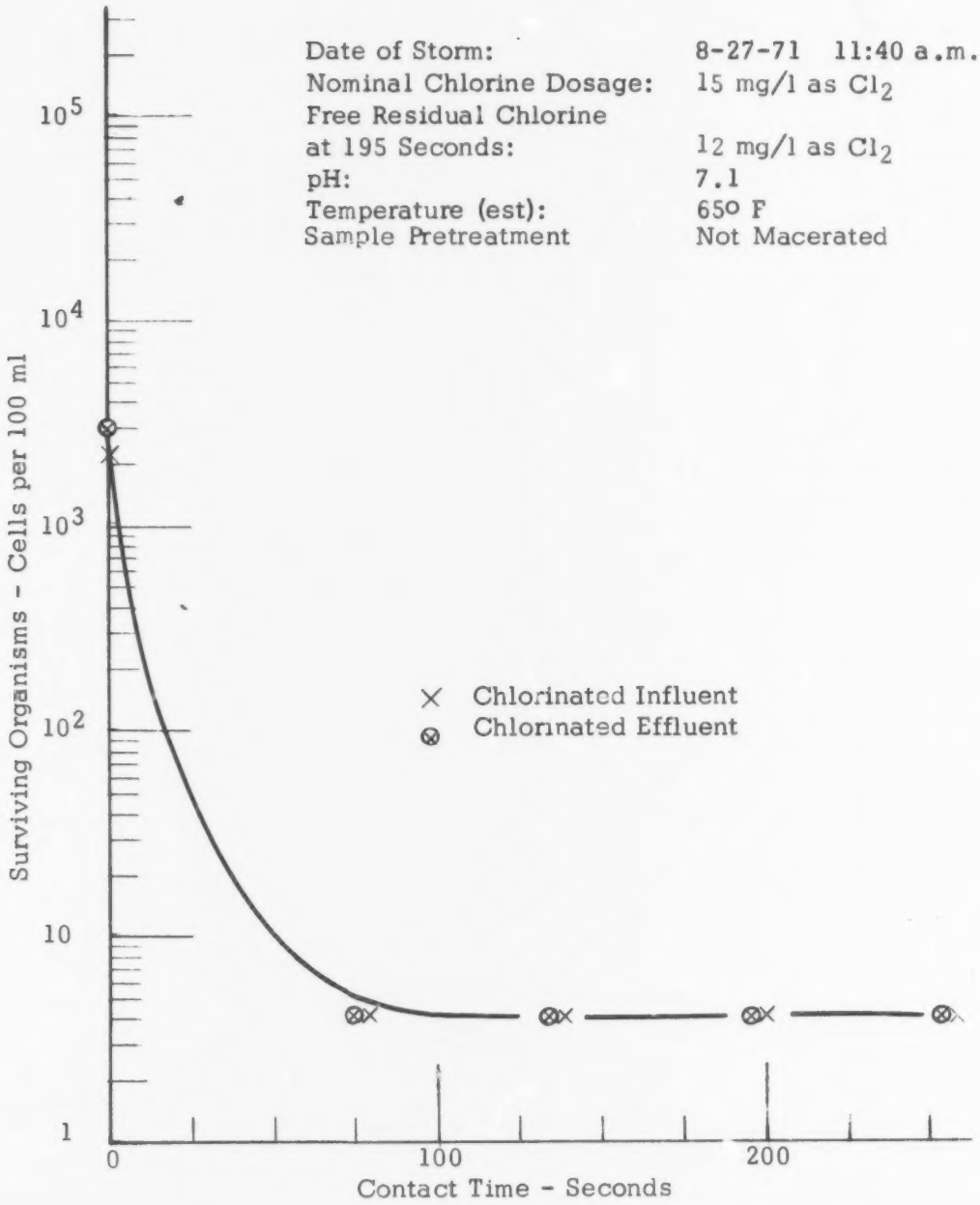

Figure 26

Fecal Coliform Survival vs Time
15 mg/l Chlorine

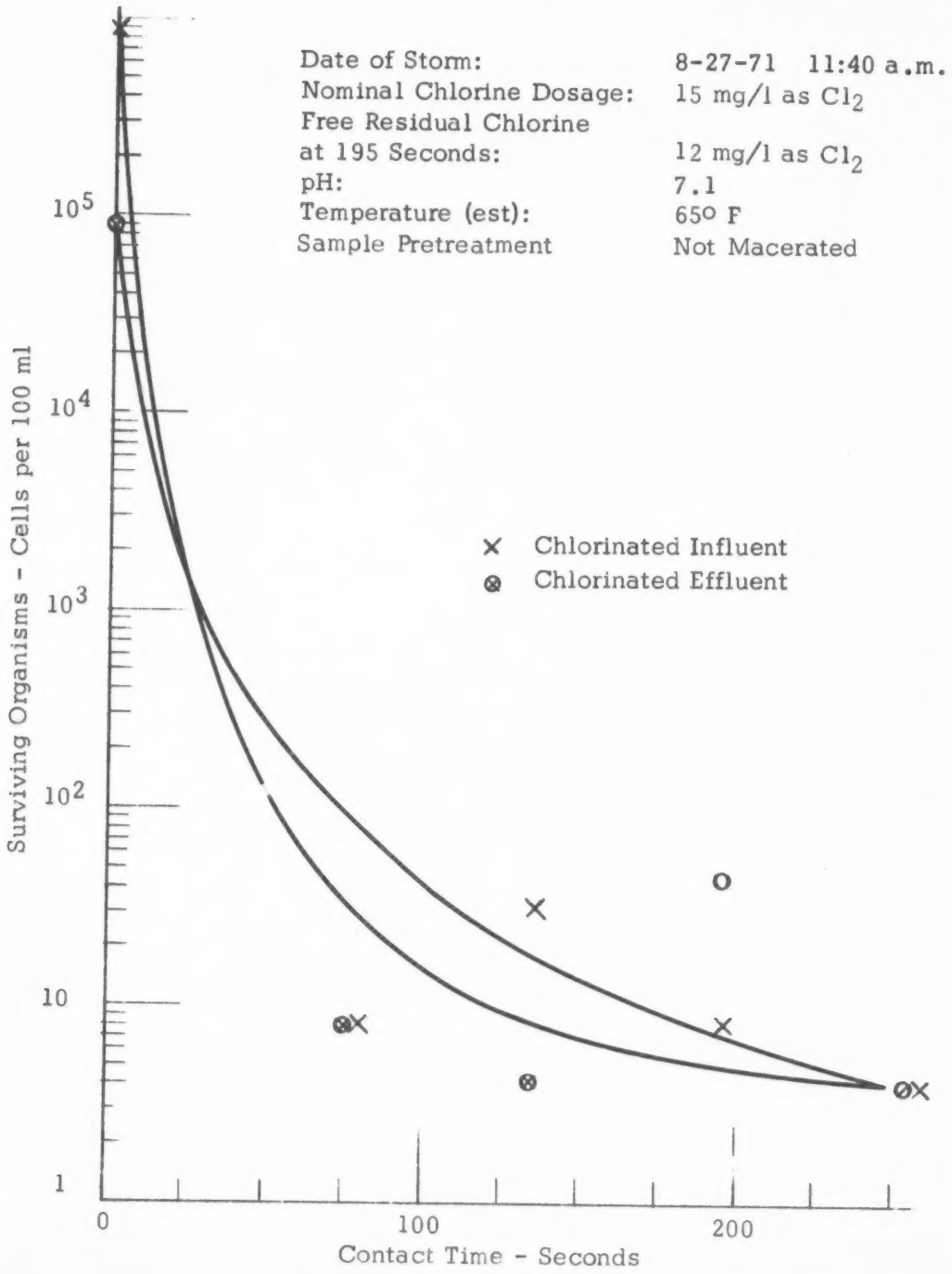

Figure 27
Total Coliform Survival vs Time
15 mg/l Chlorine

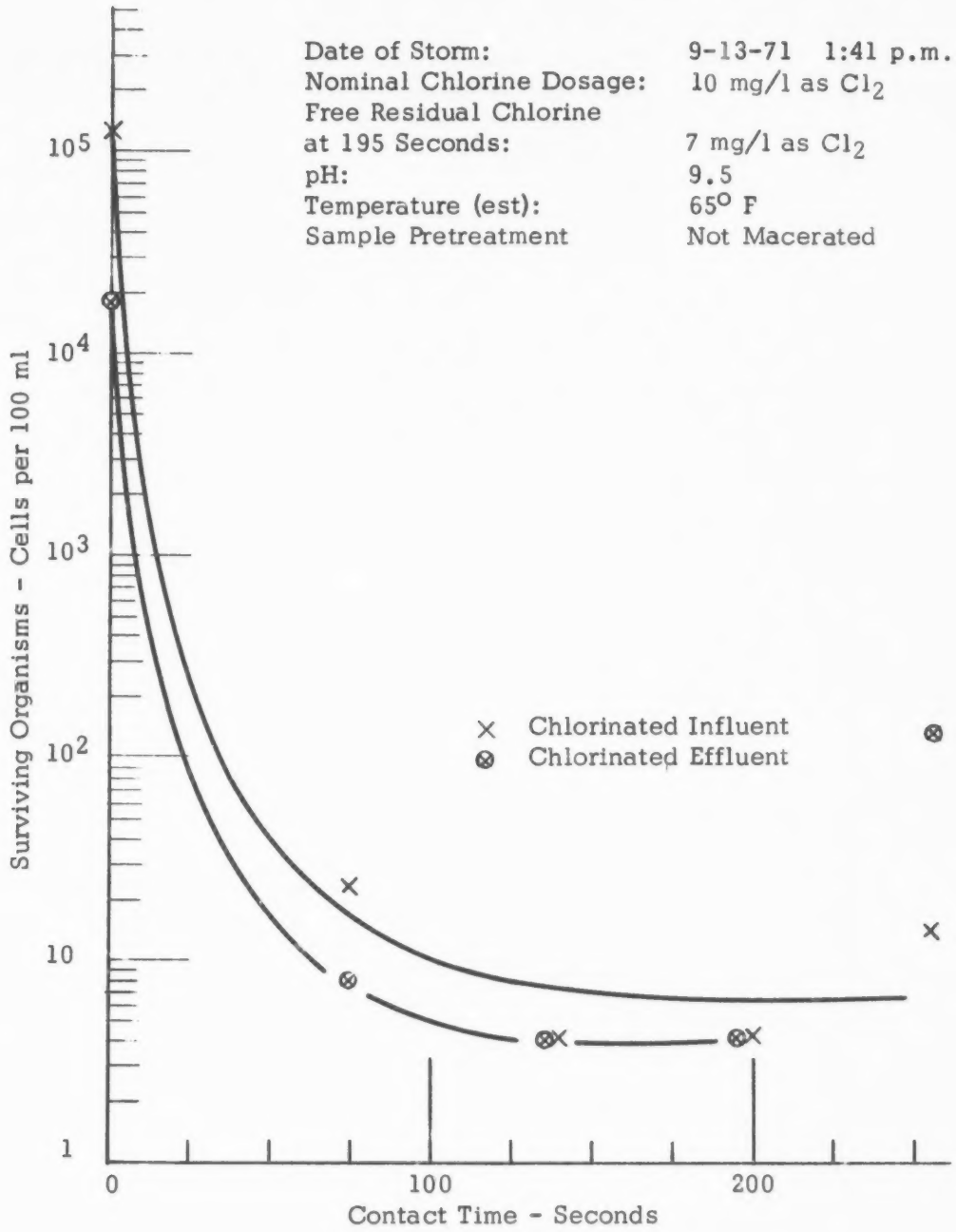

Figure 28
Fecal Coliform Survival vs Time
10 mg/l Chlorine

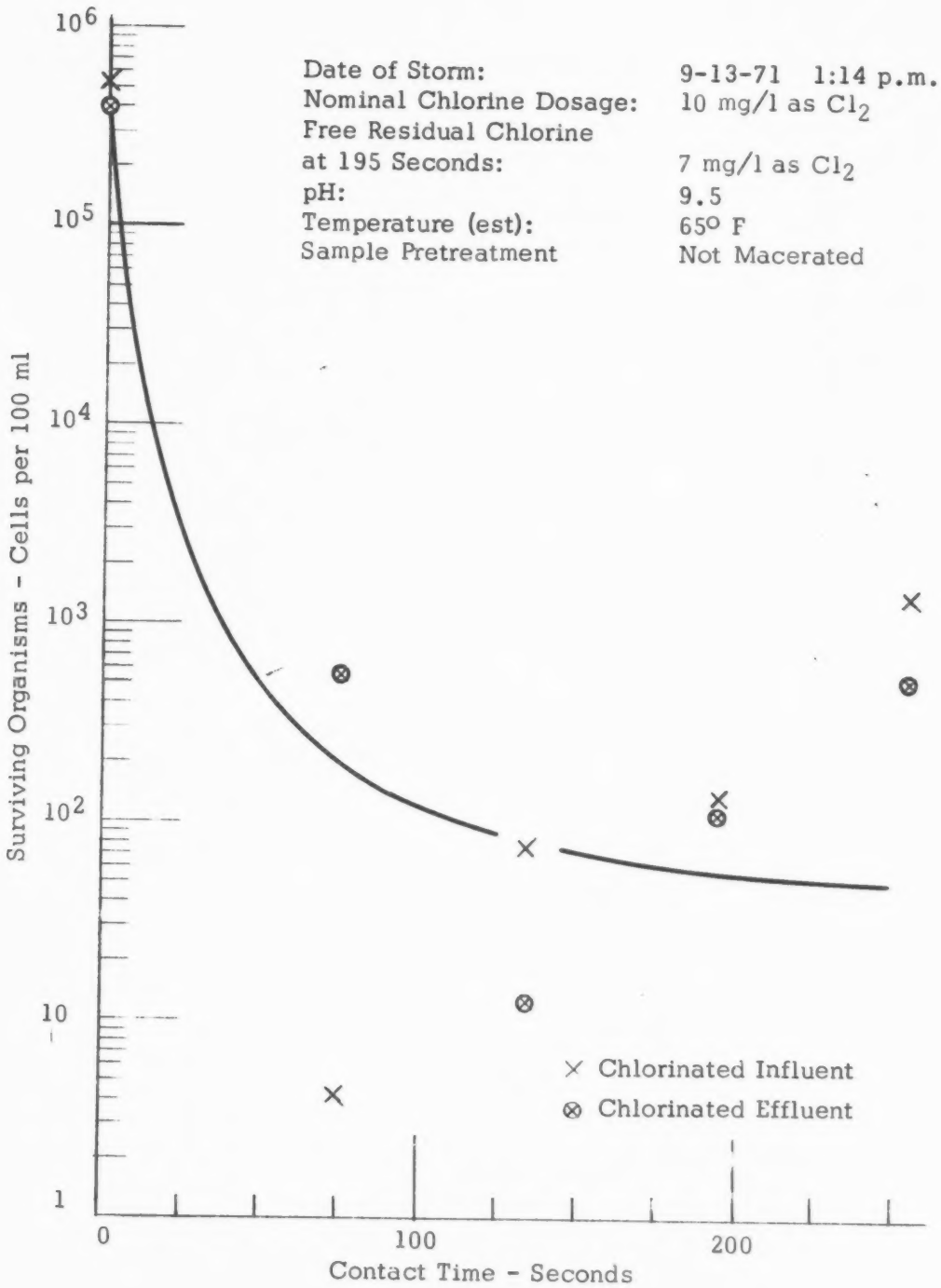

Figure 29
Total Coliform Survival vs Time
10 mg/l Chlorine

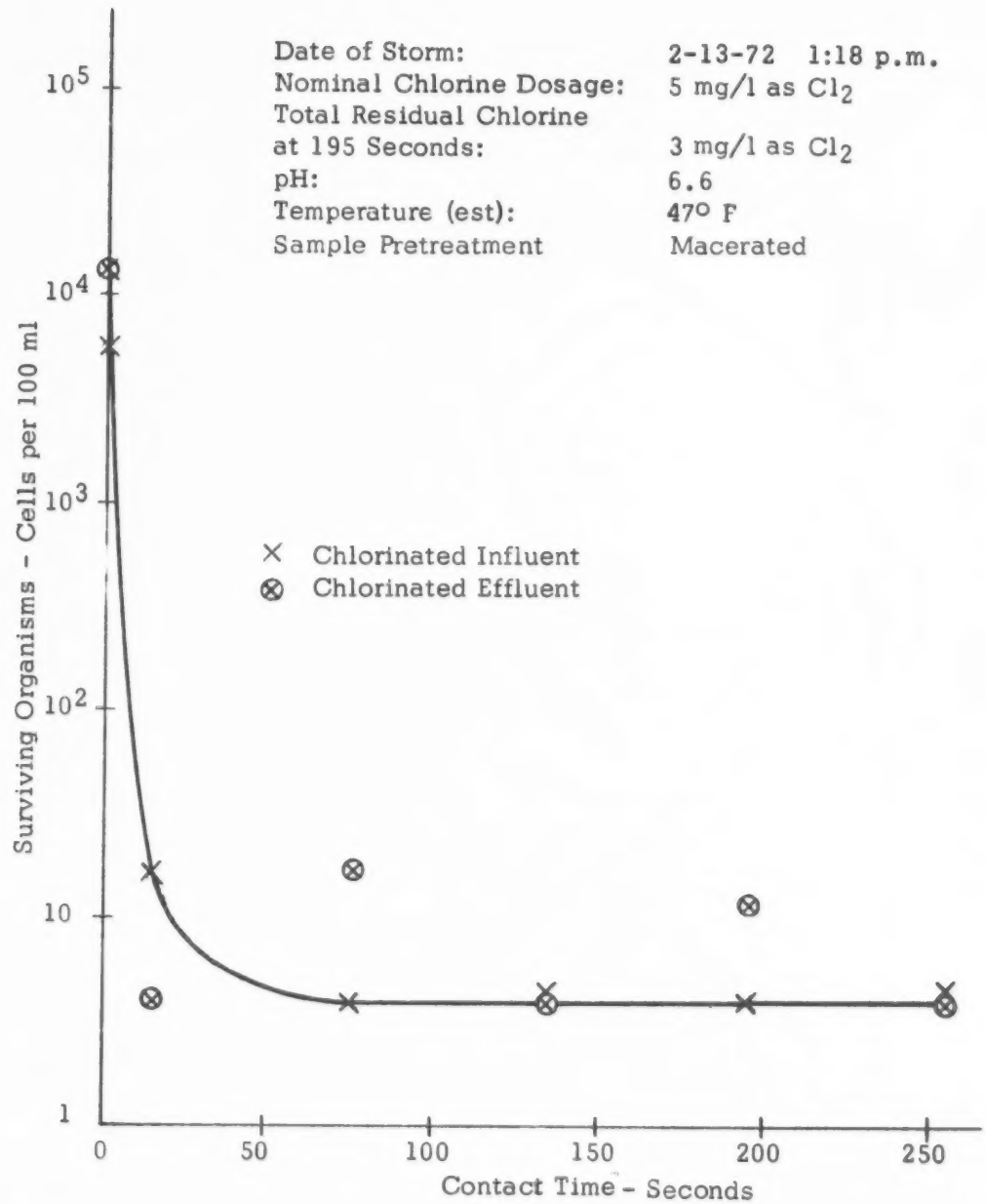

Figure 30

Fecal Coliform Survival vs Time
5 mg/l Chlorine

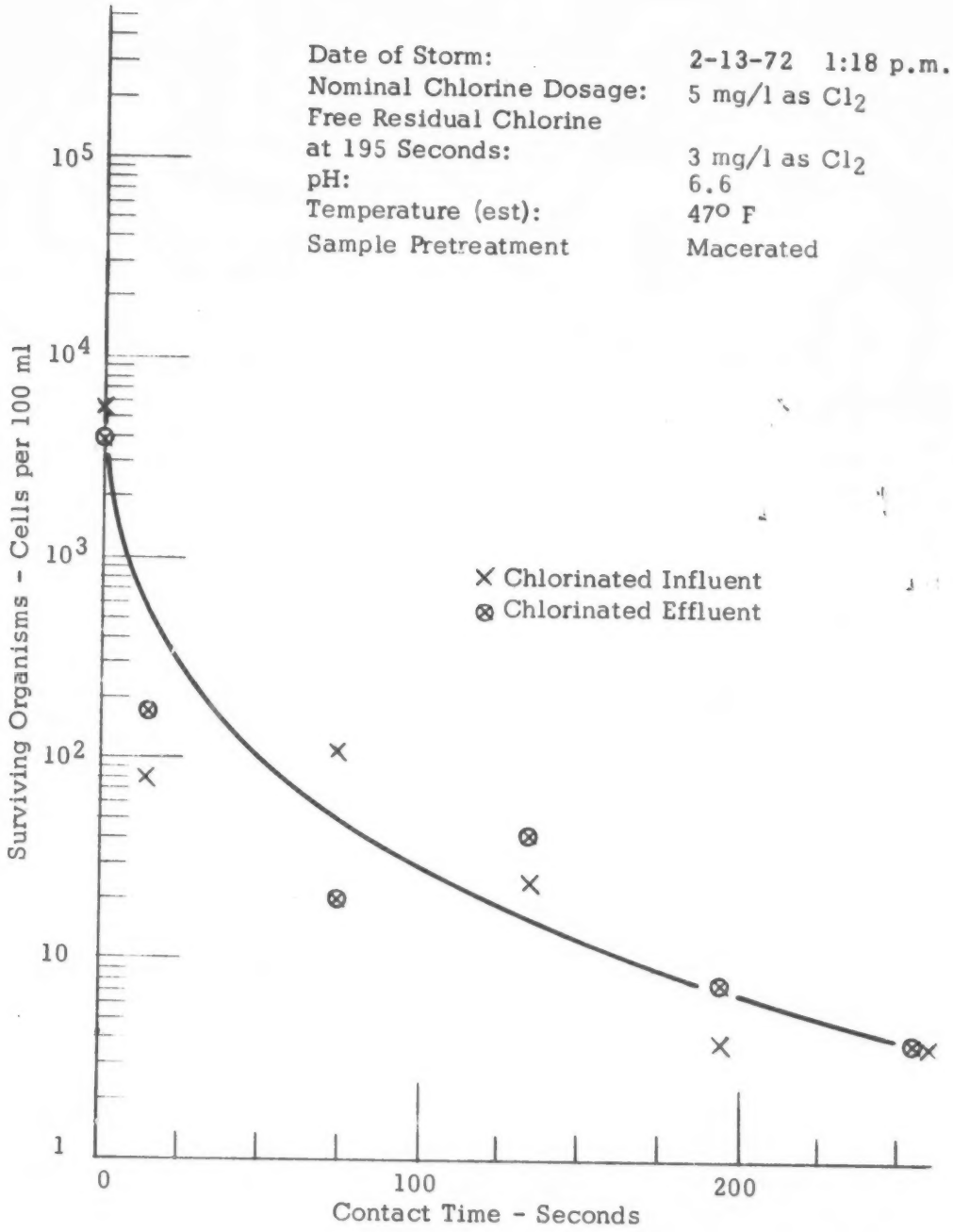

Figure 31

Total Coliform Survival vs Time
5 mg/l Chlorine

A third study made on collected effluent water only using 5 mg/l chlorine dosage is shown on Figure 30 and Figure 31. The 3 minute residual in this case was 3 mg/l.

In this study, an aliquot of each of the five varying residence samples was intensely blended before analysis in order to determine whether some organisms may be occluded by remaining solids. This effect might be more apparent on chlorinated influent samples. However, influent water could be disinfected only when an operator was present during the actual storm and this was not always possible in the case of shorter duration storm events.

The coliform counts for the other disinfection studies at 15 and 10 mg/l chlorine dosage were made on samples without prior maceration. It is unfortunate that the coliform counts for all studies were not made on both macerated and untreated samples.

Note as illustrated in Figure 32, that in each run the coliform levels drop rapidly to their minimum within the first 2 minutes of contact. Interestingly, the absolute number of surviving organisms is the same in each case, though the fractional survival may vary with influent quality.

The disinfection phase of this project as outlined in the grant Statement of Work was to apply sodium hypochlorite and ozone singly and in combination. Also, the objective disinfection level as stated above was to achieve 99.9% kill with influent total coliform levels of 100,000 cell/100 ml or greater and 99.5% kill with influent levels of 100,000 cell/100 ml or less; i.e., to achieve effluent levels of 100 to 500 cell/mg total coliform or less. As previously stated rain fall events during the contact period allowed only three disinfection studies and these three studies were performed with sodium hypochlorite dosages of 15, 10, and 5 mg/l available chlorine. As can be seen by Figures 27, 29, and 31, the disinfection performance in the high gradient chambers was much better than anticipated and total coliform levels of 10 cells/100 ml and less were achieved in every case. The minimum dosage to achieve the target disinfection level of 100 cell/100 ml total coliform was not established beyond the finding that it is less than 5 mg/l at the conditions used; i.e., 50 sec at $G = 40\ sec^{-1}$, pH 6.6 and temperature of 47° F.

Further studies should be planned at higher velocity gradients and at lower chlorine dosages. The studies at lower residual chlorine values may indicate differences in disinfection rates between microstrainer influents and effluents. Maceration of test samples prior to assay is likely to aid in revealing these differences.

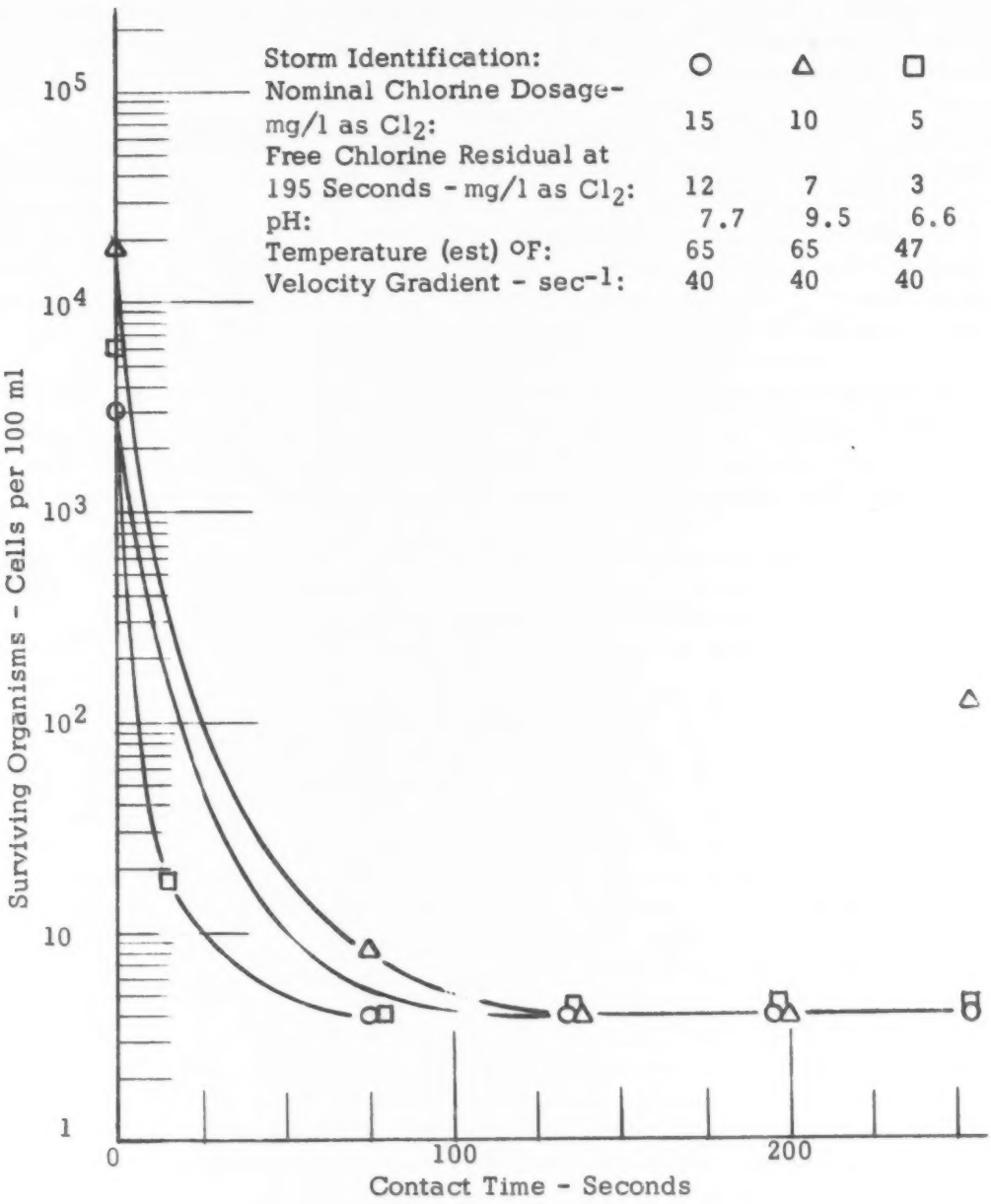

Figure 32

Comparison of Effluent Disinfection Curves
5, 10, 15 mg/l Chlorine

The disinfection studies were all conducted at a constant design flow rate. Our chambers are equipped, as are the conventional contact chambers, with a horizontal outlet weir located at an elevation of approximately 10% below the liquid level at design flow. Thus, at flows less than design rate the residence time would be increased and the velocity would be decreased almost proportionately to the flow rate reduction.

Studies (22) have shown that the velocity gradient varies as the $(\text{velocity})^{1.5}$ when the effect of velocity on the slope term is considered. In chambers with a horizontal outlet weir located near the working liquid level at design rate, the constant time will vary inversely with the velocity. The net effect is that the GT product will be reduced at less than peak design flow and chamber disinfection performance can be expected to be reduced. The use of a Sutro weir, if head is available - to maintain a level proportional to flow; i.e., constant velocity and constant residence time, and thus a constant GT product are proposed (22).

SECTION VIII

GLOSSARY

Boucher's Filtrability Index

The strainability of a specific water with suspended solids, through a specific screen as defined by the equation

$$\ln (H/Ho) = IV$$ (See Figure 13)

Break Point

The differential pressure across a specific screen with deposited solids which is sufficient to rupture the bridge of solids across the screen opening. It indicates the limiting thickness of solids for a given flow rate and screen size. (See Figure 13)

Combined Sewer Overflow

The purposeful discharge of the contents of a combined sewer to a receiving stream. The contents of a combined sewer at these times is a mixture of stormwater and sanitary wastes. During dry weather the contents of a combined sewer are exclusively sanitary flow. This flow is normally directed to a sewage plant by a regulator adjusted to pass 1.5 - 3.0 times the mean dry weather flow, or on more sophisticated regulators, the residual capacity of the interceptor sewer or sewage plant. Combined sewer flows, which because of rain fall are in excess of the above, are discharged to the stream.

Maceration

A pretreatment for samples undergoing analysis. It consists of intense mixing-shearing at low temperature and is generally performed in a Waring Blender. Detail of maceration technique appears in Appendix A.

Membrane Refiltration Test

A measure of the filtrability of suspended solids. It is normally used as an analytical test to evaluate the effect of suspended solids particle characteristics.

Mixing Intensity

The degree of turbulence created by the expenditure of mechanical and

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hydraulic energy. It is proportional to the rate of fresh liquid to liquid interface created. The unit of mixing intensity is the velocity gradient.

Permeability

The flow rate in gpm-cp/ft^2 promoted through a granular bed by a differential pressure equal to one foot of liquid head per foot of bed thickness. (cp = viscosity in centipoise)

Permeability Parameter

As applied to Microstraining, where flow rates and mat thicknesses are variable within the filtering cycle, the permeability parameter is the average flow rate in (gpm/ft^2) (centipoise) through the final mat thickness deposited on the screen. The thickness of the final mat is calculated from the weight of suspended solids deposited and a bulk density which is estimated or measured in an Imhoff cone.

Plug Flow

The passage of liquid through a chamber such that all increments of liquid move only in the direction of flow and at equal velocity.

Runoff Coefficient

The fraction of the flow calculated to have reached the ground from rain gauge data which reaches the sewer. The coefficient may be measured from actual data or estimated from the topography of the drainage area.

Theoretical Residence Time

In a dynamic system, Theoretical Residence Time is the duration any increment of liquid is within the boundaries of a system. It is calculated from the liquid volume of the system divided by the volumetric flow rate of the liquid.

When batch systems are used to simulate flow systems as in a beaker-type disinfection study, the stirring time is sometimes referred to as Theoretical Residence Time.

Velocity Gradient

The unit measure of mixing intensity. It is defined as the difference in velocity of two parallel planes of fluid divided by the distance between the two planes, and has the dimensions of $\frac{ft/sec}{ft}$.

SECTION IX

ACKNOWLEDGMENTS

This work was supported by the Environmental Protection Agency and the City of Philadelphia.

The overall planning and execution was under the general direction of Richard Field, Chief, Storm and Combined Sewer Technology Branch, EPA, Carmen F. Guarino and Joseph Radzuil, Co-Project Leaders for the City of Philadelphia, William Wankoff, Co-Project Engineer for the City, and William Keilbaugh, Manager of Crane-Environmental Systems Division's R&D Department.

Thanks are extended to Martin Lozanoff, of the City of Philadelphia's Northeast Water Pollution Control Laboratory and Paul Baker, Chemist, who performed all of the analyses without which this work would have not been possible. Thanks are also extended to the personnel of the City of Philadelphia Water Department's R&D Group, who transported the samples and tended the rain gauge.

Valuable contributions to the initial planning of the disinfection work were received from C. W. Chambers, Robert Smith, and E. E. Geldrich, of the EPA Advanced Waste Treatment Research Laboratory in Cincinnati, Ohio.

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SECTION XI

PUBLICATIONS

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No patents have been produced or applied for under this project.

SECTION XII

APPENDICES

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Appendix A

Microstrainer Project: Analytical Data

This Appendix was prepared February 2, 1972, by Martin Lozanoff, Head of the Central Water Pollution Control Laboratories of the City of Philadelphia, as a memorandum to William Wankoff, Co-Project Engineer for the City of Philadelphia.

To assist in interpreting and evaluating the analytical data generated during the Microstraining project, it is appropriate to review some of the procedures practiced by the laboratory to insure valid results. These procedures have two main objectives: first, to control the quality of the results and second, to measure the quality of the results.

Procedures to control quality are applied to every phase of the analytical work load including sampling techniques, sample preservation, aliquotting procedures, dilution procedures, chemical or physical separations and purifications, instrumental procedures, calculation and reporting of results, etc. They take such forms as the use of standard analytical methods; the production of high purity water; the periodic servicing and adjusting of analytical balances; the use of filters on compressed air lines to trap oil, moisture, and other contaminants; the use of constant voltage transformers to minimize line voltage variations; the choice of appropriate as well as properly cleaned glassware; the use of high purity reagents, solvents, and gases, their proper storage and correct selection for particular applications; the calibration and standardization of analytical instruments; the proper use of these instruments and care of the many accessories; the thorough and continual training of personnel; and countless other measures.

Procedures to measure analytical quality consist of daily checks on precision and accuracy and comparing them to predetermined limits. Precision is determined by duplicate analyses and accuracy, where applicable, by spiked or standard samples. Fully 20%, and frequently more, of an analyst's time is spent checking precision and accuracy. Following is a brief discussion of a few of the analytical considerations involved in each set of samples from the microstrainer project.

Analytical Methods

Unless otherwise specified, all analyses are run according to the procedures set forth in Standard Methods, 12th edition. If modifications are made, such changes are clearly stated. When procedures are used that are not contained in Standard Methods, the details and source of such procedures are given. The emphasis is on standardization of methods

throughout the laboratory using a reference that is widely accepted and, in fact, specified by Pennsylvania State Law. (The EPA Methods Manual is used as a comparative reference.)

Pure Water

The laboratory utilizes the Millipore Super - Q (Super Quality) system to produce high purity water. This system consists essentially of disposable cartridges for prefiltration, organic adsorption, deionization, and 0.45 micron filtration. Water up to 18 megohms resistance is produced. The distillate from an ordinary still is only 0.5 megohms.

The output from the Super - Q system is continuously monitored for purity by a built-in resistivity meter. In addition, periodic checks are made on a separate conductivity meter to verify the Super - Q meter reading. Moreover, frequent checks on the organic quality of the effluent are made by total organic carbon analyses.

A Super - Q system is installed on each floor with an outlet located in each lab. Use of this system eliminates storage and transportation of distilled water and the consequent likelihood of contamination.

pH

The electrometric method is used. Prior to use the pH meter is standardized with three buffer solutions and after use with one buffer solution in the range of analyses.

Specific Conductance

1. For each set of samples the cell constant is determined with a standard solution.

2. Every fifth sample is a duplicate; at least one duplicate is run with each set of samples.

3. Periodically, duplicate readings are taken with two different cells.

4. All readings are reported at 25° C.

Total Suspended Solids

1. The method used is filtration with Gooch crucibles using glass fiber filter discs.

2. Every fifth sample is a duplicate with at least one duplicate for each set of samples.

3. There is no satisfactory procedure for determining the accuracy of this method since the true concentration of suspended matter is unknown.

Total Dissolved Solids

1. Total dissolved solids are obtained by difference between the residue on evaporation (total solids) and total suspended solids. This method is preferred to direct evaporation of the filtrate from the total suspended solids determination in order to minimize the effect of experimental error.

2. Every fifth sample is a duplicate with at least one duplicate for each set of samples.

3. Accuracy is not measurable because there is no standard of comparison.

Biochemical Oxygen Demand

1. Three different dilutions with duplicates of each dilution are set up for each sample.

2. The BOD of each sample is reported as the average of the set of duplicate dilutions that meets the criteria set forth in Standard Methods.

3. Every tenth sample is a standard glucose - glutamic acid solution. At least one standard solution is analyzed with each set of samples.

Chemical Oxygen Demand

1. Each sample is done in duplicate and reported as the average.

2. Every tenth sample is a standard potassium acid phthalate solution. At least one standard solution is run with each set of samples.

Total Organic Carbon

1. Carbon dioxide free water for standard solutions, blanks, and dilutions is produced by the Millipore Super - Q system. Analysis of this water on the TOC analyzer at the highest sensitivity in our

working range indicates no discernible carbon.

2. Samples containing particulate matter are macerated at 23,000 rpm for a minimum of 5 minutes in a Waring blender.

3. Bore size of the syringe used for sample injection is 154 microns which is large enough to accommodate all the particles of a thoroughly macerated sample.

4. Three consecutive peaks reproducible within 1% of full scale are required for each sample.

5. Every fifth sample is either a duplicate to measure precision or a spiked sample to measure accuracy. If less than five samples are analyzed, then at least one duplicate and one spiked sample are run.

Total and Fecal Coliform

1. Three different sample volumes are filtered for each sample.

2. At least two colony counts are made of each filter.

Rate Coefficient and Ultimate BOD

1. The procedure is outlined below.

2. The rate constant K and the ultimate BOD L are calculated by three different methods:

   a. Reed-Thereault
   b. Log - Difference
   c. Thomas Slope

3. Reported results are calculated by the Reed-Thereault method because it has the lowest standard error. K is reported to the base e on a daily basis.

Procedure for Rate Constant Analysis

1. Set up sample dilution according to the standard method.

2. Select dilutions that will result in a significant dissolved oxygen depletion in 24 hours.

3. Determine dissolved oxygen concentration daily.

4.* If more than half of the available dissolved oxygen has been depleted reaerate the sample using a reaeration adaptor and an empty BOD bottle.

5. Record the dissolved oxygen concentration before and after reaeration.

6. Multiply the dissolved oxygen consumed during each period by the dilution factor.

*Reaeration Procedure

1. The reaeration adaptor is a short piece of PVC pipe which has a taper on each end which fits a BOD bottle.

2. Remove the stopper from the bottle to be reaerated and insert one end of the reaeration adaptor.

3. Invert a clean BOD bottle and place it on the other end of the adaptor.

4. Tilt the assembly so that 1/4 to 1/2 of the dilution flows into the clean BOD bottle.

5. Shake the assembly vigorously so that the dilution is reaerated.

6. Set the assembly down in the same position it had been before it was picked up.

7. When the BOD dilution has drained into the BOD bottle remove the aeration adaptor and the second bottle.

8. Replace the stopper in the BOD bottle.

Procedure to Macerate a Sample

The maceration procedure for preparation of samples prior to determining BOD, COD, TOC and coliform counts was as follows:

1. Cool sample to 40° F.

2. Cool container of maceration device (Waring Blender) by means of ice cubes wrapped in a towel for 15 minutes.

3. Transfer cool sample to cooled blender container.

4. Blend at high speed (frappe setting) for 60 seconds.

5. Proceed at once with measurement.

The standard deviation of the precision (C_V) of the tests based upon multiple analyses of stormwater samples described above are listed in the following table under "Northeast". Also, shown for comparison are the deviation values obtained by surveys of many laboratories conducted by the Standard Methods Committee and by the EPA.

Precision & Accuracy

TEST	NORTHEAST	STD. METHODS	EPA
BOD_5	C_V = 9.0% Conc = 5-129 mg/l	C_V = 17% Conc = 184 mg/l	C_V = 20.6% Conc = 194 mg/l
COD		C_V = 6.5% Conc = 200 mg/l	C_V = 10.2% Conc = 270 mg/l
		Recovery = 95-100%	Recovery = 96%
TOC	C_V = 1.9% Conc = 16-182 mg/l	C_V = 5-10% Samples with solids	Recovery = 85% Conc = 4.9 mg/l
	Recovery = 97% Conc = 61-193 mg/l	C_V = 1-2% No Solids	Recovery = 99% Conc = 107 mg/l
SS	C_V = 5.7% Conc = 3-52 mg/l	C_V = 33% Conc = 15 mg/l	
		C_V = 10% Conc = 242 mg/l	

Coliform: C_V = 25%

Ref: "MPN & MF Comparison", Sewage & Industrial Wastes, 31, 1, 78 (Jan. 1959).

Appendix B

Organic Removal Performance

Date of Storm	Radial Flow Rate[C]	S.S. Infl mg/l	SVS In mg/l	SVS Out mg/l	SVS Removal %	DVS In mg/l	DVS Out mg/l	DVS Removal %
7- 1-71	46.0	165	ND	ND	-	ND	ND	-
7- 9-71	35.0	ND	ND	ND	-	ND	ND	-
7-19-71[B]	46.5	47	6	16	Incr	41	108	Incr
	46.5	47	-	-	-	-	-	-
	46.5	317	249	27	90	-	-	-
7-29-71[B]	45.0	27	22	12	45	34	67	Incr
	45.0	27	-	-	-	-	-	-
	45.0	27	-	-	-	-	-	-
7-30-71[A]	18.0	ND	ND	ND	-	ND	ND	-
	37.4	115	29	4	87	18	16	15
	37.4	115	-	-	-	-	-	-
	37.4	72	18	7	69	-	-	-
	50.0	39	21	14	33	-	-	-
	50.0	11	6	16	Incr	-	-	-
7-31-71	42.0	52	24	15	38	-	-	-
8-27-71	47.0	66	17	1	94	100	57	43
	47.0	66	-	-	-	-	-	-
	47.0	66	-	-	-	-	-	-
	40.0	29	16	6	63	10	17	Incr
	40.0	29	-	-	-	-	-	-
	40.0	29	-	-	-	-	-	-
8-28-71	37.0	68	36	1	97	19	25	Incr
	37.0	68	-	-	-	-	-	-
	37.0	68	-	-	-	-	-	-
9-10-71	13.5	190	49	17	65	48	98	Incr
	13.5	-	-	-	-	-	-	-
	13.5	-	-	-	-	-	-	-

Sample Procedure:

[a]macerated composite
[b]discrete
[c]filtered

ND - Not Determined

Remarks:
A - Sample dumped
B - Effluent conductivity 350 close to city water; influent cond. 100
C - gpm/ft^2

Appendix B

Organic Removal Performance

5 Day BOD			COD			TOC			
In	Out	Removal	In	Out	Removal	In	Out	Removal	
mg/l		%	mg/l		%	mg/l		%	
ND	ND	-	ND	ND	-	ND	ND	-	
ND	ND	-	ND	ND	-	ND	ND	-	
ND	ND	-	720	2880	Incr	ND	ND	-	
ND	ND	-	ND	ND	-	212	1048	Incr	a
ND	ND	-	-	-	-	-	-	-	b
30	30	no change	79	68	14	ND	ND	-	
20	ND	-	-	-	-	-	-	-	c
28	28	no change	89	108	Incr	39	29	25	a
ND	ND	-	ND	ND	-	ND	ND	-	
260+	252+	-	2350	6270	Incr	ND	ND	-	
348	256+	-	2160	10200	Incr	864	3154	Incr	a
ND	ND	-	-	-	-	-	-	-	b
ND	ND	-	-	-	-	-	-	-	b
ND	ND	-	-	-	-	-	-	-	b
ND	ND	-	-	-	-	-	-	-	b
ND	165	ND	150	310	Incr	ND	ND	-	
15	175	Incr	300	320	Incr	34	80	Incr	a
ND	170	ND	-	-	-	-	-	-	c
880	550	38	3760	1830	52	ND	ND	-	
940	440	53	1410	990	30	409	224	45	a
1180	540	54	-	-	-	-	-	-	c
2650	940	65	4680	1740	63	ND	ND	-	
2700	1150	58	5230	1920	63	1278	504	60	a
2000	ND	-	-	-	-	-	-	-	c
820	ND	-	1820	8350	Incr	ND	ND	-	
920	3300	Incr	1820	7260	Incr	470	1950	Incr	a
880	2550	Incr	-	-	-	-	-	-	c

Sample Procedure:

[a]macerated composite
[b]discrete
[c]filtered

ND - Not Determined

Appendix B

Organic Removal Performance

Date of Storm	Radial Flow Rate[C]	S.S. Infl mg/l	SVS In mg/l	SVS Out mg/l	SVS Removal %	DVS In mg/l	DVS Out mg/l	DVS Removal %
9-11-71	15.2	90	29	16	45	24	51	Incr
	15.2	90	-	-	-	-	-	-
	15.2	130	44	8	66	19	61	Incr
	15.2	130	-	-	-	-	-	-
	14.5	80	32	20	38	19	79	Incr
	14.5	80	-	-	-	-	-	-
	15.0	96	32	24	25	16	89	Incr
	15.0	96	-	-	-	-	-	-
	-	347	110	14	87	-	-	-
	-	223	42	ND	-	-	-	-
9-12-71	-	93	20	14	87	ND	ND	-
9-13-71	38.0	231	78	6	93	-	-	-
	36.5	72	24	17	29	ND	ND	-
	36.5	72	-	-	-	-	-	-
9-21-71	32.5	33	18	10	45	-	-	-
	41.0	64	26	18	31	23	26	Incr
	41.0	64	-	-	-	-	-	-
	41.0	64	-	-	-	-	-	-
10-10-71	34.0	751	204	0	100	ND	ND	-
	34.0	751	-	-	-	-	-	-
	34.0	751	-	-	-	17	68	Incr
	34.0	330	81	3	96	ND	ND	-
	34.0	330	-	-	-	-	-	-
	34.0	330	-	-	-	28	39	Incr
	46.5	14	7	4	43	ND	ND	-
	46.5	14	-	-	-	-	-	-
	46.5	14	-	-	-	11	16	Incr

Sample Procedure:

[a]macerated composite
[b]discrete
[c]filtered

ND - Not Determined

Remarks:
A - Sample dumped
B - Effluent conductivity 350 close to city water; influent cond. 100
C - gpm/ft^2

Appendix B

Organic Removal Performance

5 Day BOD			COD			TOC			
In mg/l	Out	Removal %	In mg/l	Out	Removal %	In mg/l	Out	Removal %	
2240	11750	Incr	3450	19770	Incr	ND	ND	-	
ND	ND	-	ND	ND	-	1200	7700	Incr	a
2280	2350	Incr	3270	3180	03	ND	ND	-	
ND	ND	-	ND	ND	-	1270	1080	15	a
1100	1350	Incr	3070	3690	Incr	ND	ND	-	
ND	ND	-	ND	ND	-	730	840	Incr	a
3300	4100	Incr	7260	4090	44	ND	ND	-	
ND	ND	-	ND	ND	-	1900	2550	Incr	a
1580+	1580+	no change	-	-	-	-	-	-	b
1580+	1580+	no change	-	-	-	-	-	-	b
1280	1380	Incr	ND	ND	-	630	640	Incr	
2150	1750	19	-	-	-	-	-	-	b
1730	1740	Incr	3930	3390	14	ND	ND	-	
2150	1250	42	4460	3570	20	1065	810	24	a
1620+	800	51	-	-	-	810	495	38	b
885	1140	Incr	2820	3360	Incr	ND	ND	-	
1050	930	11	2650	3270	Incr	850	775	9	a
1640+	1640+	no change	-	-	-	-	-	-	c
1910	86	95	3840	392	90	ND	ND	-	
1940	82	96	3440	440	87	980	47	95	a
1950	70	97	-	-	-	-	-	-	c
129	112	13	256	216	16	ND	ND	-	
112	98	13	280	248	11	78	54	31	a
106	124	Incr	-	-	-	-	-	-	c
8	7	13	32	200	Incr	ND	ND	-	
7	7	no change	384	416	Incr	10	10	no change	a
7	7	no change	-	-	-	-	-	-	c

Sample Procedure:

[a]macerated composite
[b]discrete
[c]filtered

ND - Not Determined

Appendix B

Organic Removal Performance

Date of Storm	Radial Flow Rate[C]	S.S. Infl mg/l	SVS In mg/l	SVS Out mg/l	SVS Removal %	DVS In mg/l	DVS Out mg/l	DVS Removal %
11- 2-71	36.4	357	76	48	37	30	32	Incr
	36.4	357	-	-	-	ND	ND	-
11-29-71	36.4	115	31	23	26	ND	ND	-
	36.4	115	-	-	-	-	-	-
	36.4	115	-	-	-	17	18	Incr
2-13-72	16.8	208	49	30	39	52	19	63
	16.8	208	-	-	-	-	-	-
	16.8	208	-	-	-	-	-	-

Sample Procedure:

[a]macerated composite
[b]discrete
[c]filtered

ND- Not Determined

Remarks:
A - Sample dumped
B - Effluent conductivity 350 close to city water; influent cond. 100
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Appendix B

Organic Removal Performance

5 Day BOD			COD			TOC			
In	Out	Removal	In	Out	Removal	In	Out	Removal	
mg/l		%	mg/l		%	mg/l		%	
55	26	53	230	170	26	77	38	51	
69	34	51	250	180	28	ND	ND	-	a
13	26	Incr	117	117	no change	ND	ND	-	
14	28	Incr	110	61	45	23	22	4	a
-	-	-	-	-	-	-	-	-	c
6	16	Incr	81	76	6	ND	ND	-	
5	15	Incr	68	41	40	24	21	13	a
2	8	Incr	-	-	-	-	-	-	

Sample Procedure:

[a]macerated composite
[b]discrete
[c]filtered

ND - Not Determined

Appendix C

Total Coliform Removal by Microstraining

Date of Storm	Radial Flow Rate gpm/ft^2	S.S. Influent mg/l	Temp °F	pH (1)	Across Microstrainer		
					Influent Cells/100 ml x 10^6	Effluent	Removal %
7- 1-71	46.0	165	NR	ND	ND	ND	-
7- 9-71	35.0	ND	NR	ND	ND	ND	-
7-19-71	46.5	47	68	7.6	0	0	-
7-29-71	45.0	27	69	7.1	2.80	0.001(4)	97.0
					3.30	0.180	94.5(2)
7-30-71	18.0	ND	NR	ND	ND	ND	-
	37.4	115	68	6.9	0.90	0.130	85.6
					0.87	0.880	no change (2)
	50.0	39	68	6.8	ND	ND	-
7-31-71	42.0	52	75	NR	ND	ND	-
8-27-71	47.0(5)	66	66	7.2	0.39	0.100(3)	74.5
	43.0			7.4	2.00	0.001(4)	99.0+(2)
	40.0	29	68	7.1	0.85	0.092	89.2
				7.3	0.73	0.550	24.7(2)
8-28-71	37.0	68	67	6.8	2.40	0.290	88.0
				6.8	4.80	1.500(3)	69.0(2)

(1) pH of filtered influent

(2) Values are results on analyses on another aliquot of same composite after intense maceration

(3) Estimated count based on non-ideal colony count

(4) Zero plate count based on 0.1 ml sample

(5) Analytical results are from a composite of two storms, one at 3:42a and one at 5:50a

ND--Not Determined
NR--Not Reported

Appendix C

Total Coliform Removal by Microstraining

Date of Storm	Radial Flow Rate gpm/ft^2	S.S. Influent mg/l	Temp °F	pH (1)	Across Microstrainer		
					Influent Cells/100 ml x 10^6	Effluent	Removal %
9-10-71	13.5	190	72	7.5	0.28	1.300(3)	Incr
				7.5	0.62	2.700	Incr (2)
9-11-71	15.2	90	69	6.9	8.40(3)	0.750	91.0
				6.6	6.10	0.990(3)	84.0(2)
	15.2	130	69	6.7	8.70(3)	0.079	99.0+
				6.9	21.00	0.072	99.0+(2)
	14.5	80	69	6.7	2.30	0.081	96.5
				7.3	6.60	0.090(3)	98.6(2)
	15.0	96	68	6.7	0.88	0.001(4)	99.0+
				7.8	0.89	0.001(4)	99.0+(2)
9-12-71	probably none	93	67	7.2	0.06(3)	0.001(4)	98.3
9-13-71	36.5	72	68	7.2	0.53	0.400	24.5
				6.7	0.78	4.200	Incr (2) (540%)
9-21-71	41.0	64	66	7.4	0.46	0.013(3)	97.2
				ND	0.24	0.039	83.8(2)

(1) pH of filtered influent
(2) Values are results on analyses on another aliquot of same composite after intense maceration
(3) Estimated count based on non-ideal colony count
(4) Zero plate count based on 0.1 ml sample

ND-Not Determined

Appendix C

Total Coliform Removal by Microstraining

Date of Storm	Radial Flow Rate gpm/ft2	S.S. Influent mg/l	Temp °F	pH (1)	Across Microstrainer		
					Influent	Effluent	Removal
					Cells/100 ml x 10^6		%
10-10-71	34.0	751	64	6.9	0.20	0.001(4)	99.0+
				7.2	0.10(6)	0.001	99.0(2)
	34.0	330	64	7.2	1.40(3)	0.004(3)	99.0+
				7.5	2.10	0.008(3)	99.0+
	46.5	14	65	6.9	0.90(3)	0.050	94.4
				7.3	1.10(3)	1.360(3)	Incr (2) (123%)
11- 2-71	36.4	357	67	7.4	0.320	0.800	Incr (250%)
					0.760	0.680	10.0(2)
11-29-71	36.4	115	50	7.3	0.570	0.780	Incr (134%)
					0.340	0.850	Incr (250%)
2-13-72	16.8	208	45	7.0	0.005	0.070	Incr (1400%)
				7.5	0.004	0.036(2)	Incr (900%) (2)

(1) pH of filtered influent
(2) Values are results on analyses on another aliquot of same composite after intense maceration
(3) Estimated count based on non-ideal colony count
(4) Zero plate count based on 0.1 ml sample
(6) Zero plate count based on 0.001 ml sample

Appendix D

Fecal Coliform Removal by Microstraining

Date of Storm	Radial Flow Rate gpm/ft^2	S.S. Influent mg/l	Temp ^oF	pH (1)	Across Microstrainer		
					Influent Cells/100	Effluent ml x 10^6	Removal %
7- 1-71	46.0	165	NR	NR	NR	NR	-
7- 9-71	35.0	NR	NR	NR	NR	-	-
7-19-71	46.5	47	66	7.6	0	0	no change
					0	0	no change[a]
7-29-71	45.0	27	69	7.1	0.0740	.00100(5)	98.7
					0.0610	.00200(2)	97.0[a]
7-30-71	18.0	NR	NR	NR	NR	NR	-
	37.4	115	68	6.8	0.0010(8)	.00010(6)	90.0
					0.0010	.00080	20.0[a]
					0.0010	.00080	20.0[a]
	50.0	39	68	6.8	NR	NR	-
7-31-71	42.0	52	75	NR	NR	NR	-
8-27-71	47.0(9)	66	66	7.2	0.0130(2)	.00003(2)	99.0+
	43.0			7.4	0.0600(3)	.00001(2)	99.0+[a]

(1) pH of filtered influent
(2) Estimated count based on non-ideal colony count
(3) At least 60 colonies in 0.1 ml sample
(4) At least 60 colonies in 1.0 ml sample
(5) At least 60 colonies in 10 ml sample
(6) Zero plate count based on 1 ml sample
(7) Zero plate count based on 10 ml sample
(8) Zero plate count based on 0.1 ml sample
(9) Analytical results are from a composite of two storms, one at 3:42a and the other at 5:50a

[a]Values are results of analyses on another aliquot of same composite after intense maceration

NR-Not Reported

Appendix D

Fecal Coliform Removal by Microstraining

Date of Storm	Radial Flow Rate gpm/ft^2	S.S. Influent mg/l	Temp ^oF	pH (1)	Across Microstrainer		
					Influent Cells/100 ml x 10^6	Effluent	Removal %
8-27-71	40.0	29	68	7.1	0.0023	.00300(2)	Incr (130%)
				7.3	0.0220	.00600(2)	73.0[a]
8-28-71	37.0	68	67	6.8	0.0031	.00600(4)	Incr (192%)
				6.8	0.0060(4)	.00060(5)	90.0[a]
9-10-71	13.5	190	72	7.4	0.0840(2)	.00010(6)	99.0+
				7.5	0.0260	.00011(2)	99.0+[a]
9-11-71	15.2	90	69	6.9	0.0500	.00400(2)	92.0
				6.6	0.0260	.01200(2)	54.0[a]
	15.2	130	69	6.7	0.0060(4)	.00200	67.0
				6.9	0.0270	.00020(2)	99.0+[a]
	14.5	80	69	6.7	0.0270	.00230	91.5
				7.3	0.0140(2)	.00060(5)	95.7[a]
	15.0	96	68	6.7	0.0120(2)	.00001(7)	99.0+
				7.0	0.0060(4)	.00001	99.0+[a]

(1) pH of filtered influent
(2) Estimated count based on non-ideal colony count
(3) At least 60 colonies in 0.1 ml sample
(4) At least 60 colonies in 1.0 ml sample
(5) At least 60 colonies in 10 ml sample
(6) Zero plate count based on 1 ml sample
(7) Zero plate count based on 10 ml sample
(8) Zero plate count based on 0.1 ml sample
(9) Analytical results are from a composite of two storms, one at 3:42a and the other at 5:50a

[a]Values are results of analyses on another aliquot of same composite after intense maceration

NR-Not Reported

Appendix D

Fecal Coliform Removal by Microstraining

Date of Storm	Radial Flow Rate gpm/ft^2	S.S. Influent mg/l	Temp ^oF	pH (1)	Across Microstrainer		
					Influent Cells/100	Effluent ml x 10^6	Removal %
9-12-71	probably none	93	67	7.2	0.0040(2)	.00001(7)	99.0+
9-13-71	36.5	72	68	7.2	0.1330(2)	.01900	85.7
				6.7	0.0018(2)	.04700	Incr[a](2600%)(2)
9-21-71	41.0	64	66	7.4	0.0030	.00001(7)	99.7
				NR	0.0033	.00001(7)	99.7[a]
10-10-71	34.0	751	64	6.9	0.0068	.00010(6)	98.5
				7.2	0.0170	.00010(6)	99.4[a]
	34.0	330	64	7.2	0.0270	.00010(6)	99.6
				7.5	9.2000	.00010(6)	99.0+[a]
	46.6	14	65	6.9	0.0170	.0200	Incr (118%)
				7.3	0.0530	.04500	15.0[a]

(1) pH of filtered influent
(2) Estimated count based on non-ideal colony count
(3) At least 60 colonies in 0.1 ml sample
(4) At least 60 colonies in 1.0 ml sample
(5) At least 60 colonies in 10 ml sample
(6) Zero plate count based on 1 ml sample
(7) Zero plate count based on 10 ml sample
(8) Zero plate count based on 0.1 ml sample
(9) Analytical results are from a composite of two storms, one at 3:42a and the other at 5:50a

[a]Values are results of analyses on another aliquot of same composite after intense maceration

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Appendix D

Fecal Coliform Removal by Microstraining

Date of Storm	Radial Flow Rate gpm/ft^2	S.S. Influent mg/l	Temp °F	pH (1)	Across Microstrainer		
					Influent Cells/100 ml x 10^6	Effluent	Removal %
11- 2-71	36.4	357	67	7.2	0.0040	.02700	Incr (670%)
11-29-71	36.4	115	50	7.3	0.0088	.01500	Incr (170%)[a]
					0.0470	.05200	Incr (110%)
2-13-72	16.8	208	45	7.0	0.0480	.07800	Incr (164%)[a]
					0.0600	.06400	Incr (107%)
					0.0140	.01500	Incr (107%)[a]

(1) pH of filtered influent

[a]Values are results of analyses on another aliquot of same composite after intense maceration

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